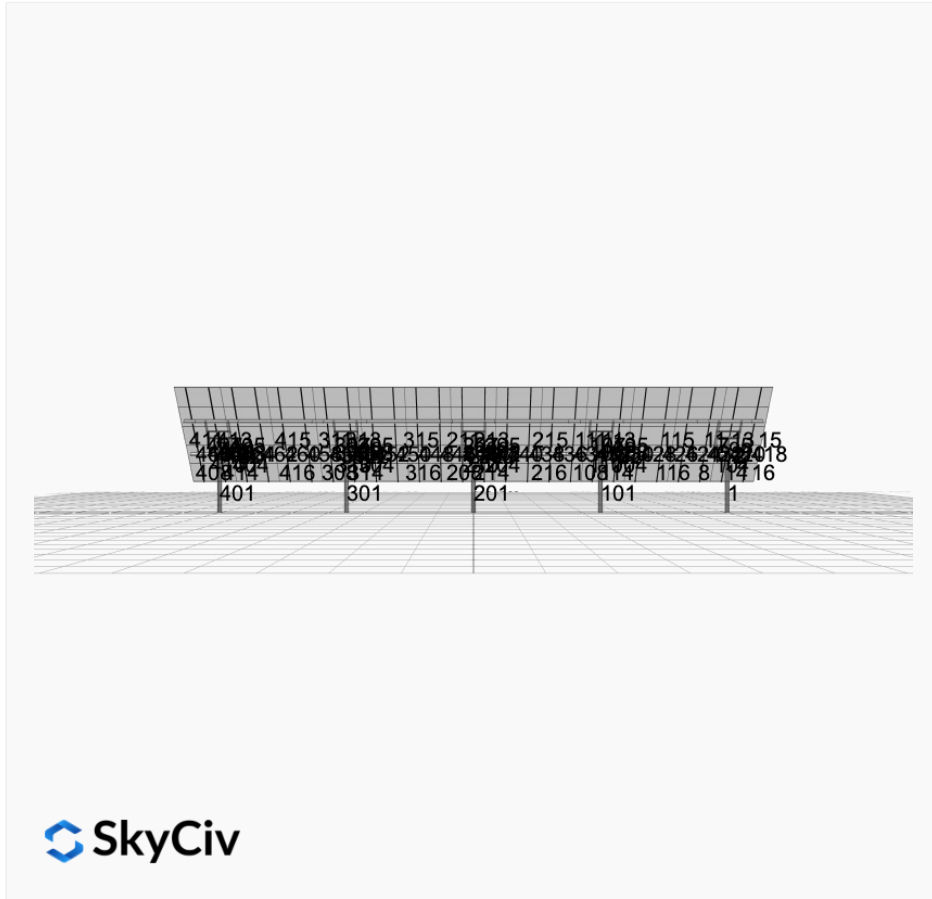


Project Name: MTSOLAR_B49GF8D71LDC - V1Jb **Date:** Tue Apr 08 2025
Location: Hagerstown, MD, USA **Number of Modules:** 65
Unique ID: 5P-22.5-8TOP-HD-24-L-5Hx13W-6II3 **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	21.58 ft
Array Dimensions E/W	102.81 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

MT Solar Bill of Materials (5P-22.5-8TOP-HD-24-L-5Hx13W-6II3)

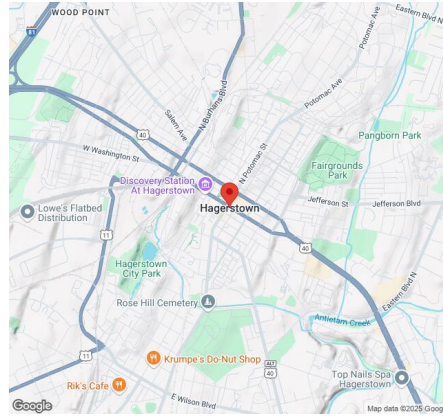
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	16
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (259in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52

Part	Qty
Ground Lug	13

Site Details:



Site Address: Hagerstown, MD, USA

Array Specification

Duty Classification:	HD
Module Width:	51.30 in
Module Length:	93.90in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	21.53 ft
Total Frame Length:	101.50 ft
Module Info/Notes:	Cs7n-715tb-ag
Array Dimensions N/S:	21.58 ft
Array Dimensions E/W:	102.81 ft
Rail Length:	259.00 in
Rail Spacing:	3.95 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	13.27 ft
Number of Poles:	5
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 8.25 ft Pile 3: 8.25 ft Pile 4: 8.25 ft Pile 5: 7.50 ft
Foundation Volume:	23.556 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Hagerstown, MD, USA
Wind Speed:	115 mph

Snow Load:

30 psf

Design Disclaimer

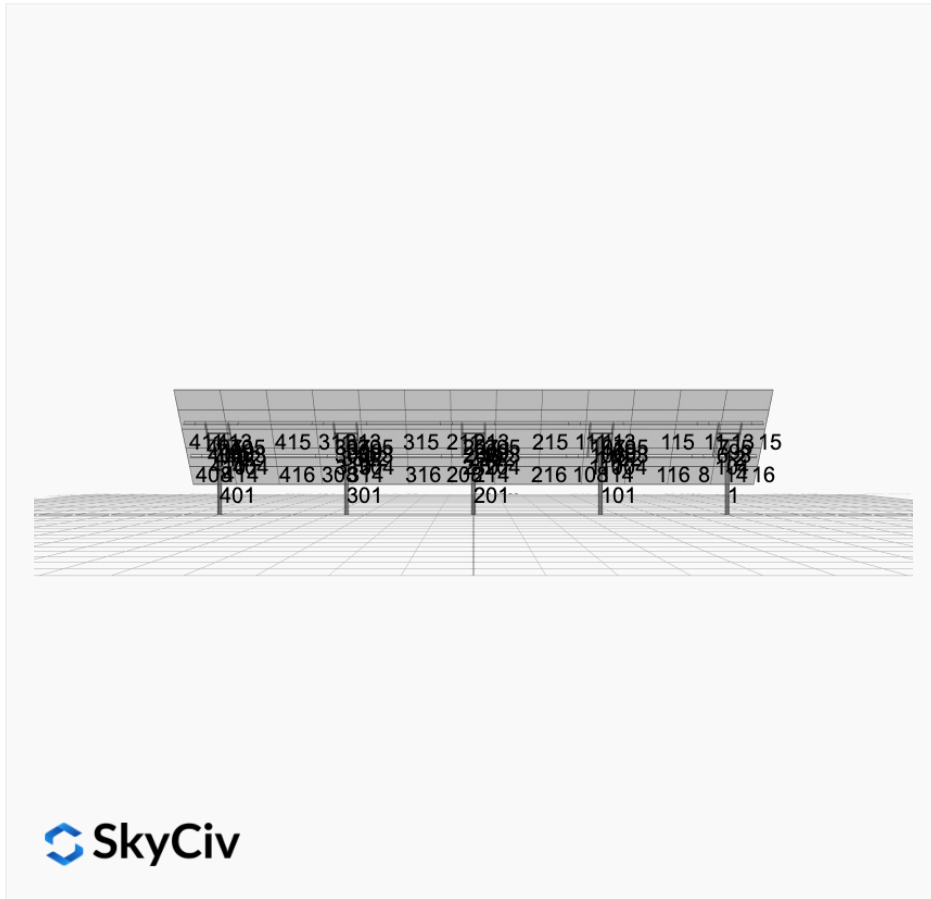
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

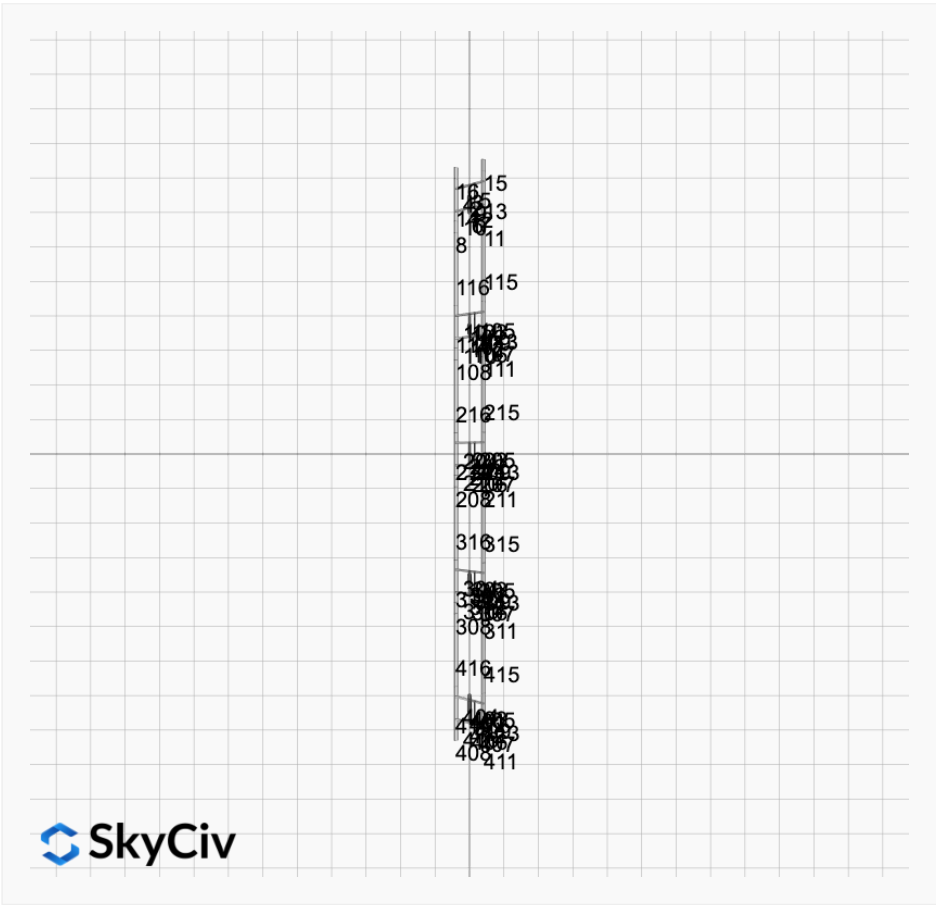
AutoDesigner Input

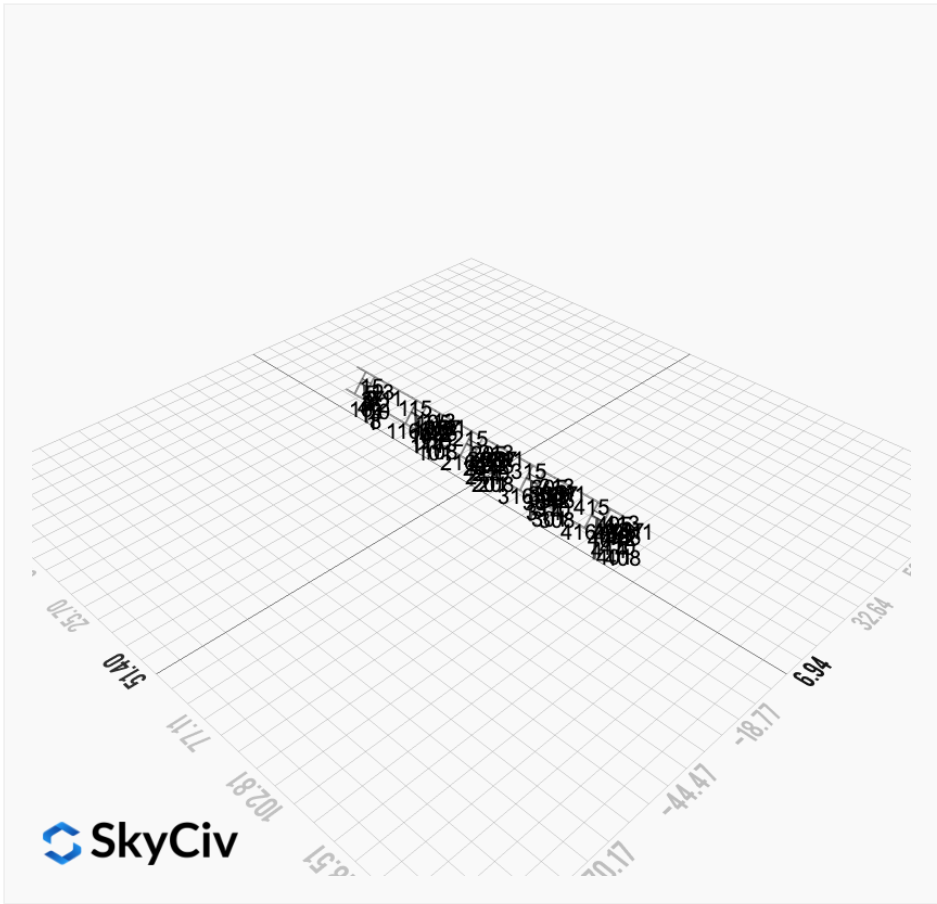
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Design Notes:

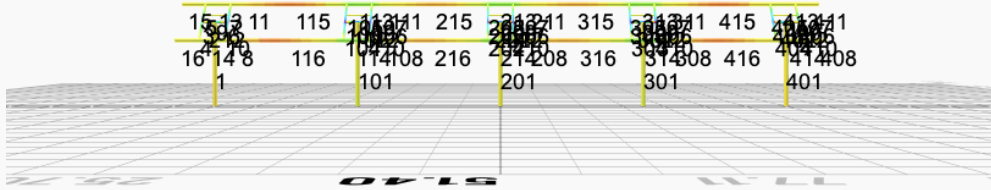
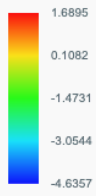
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



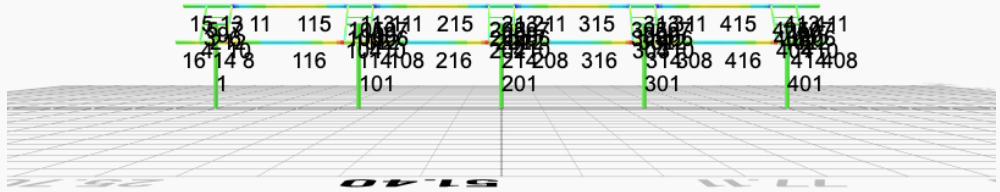




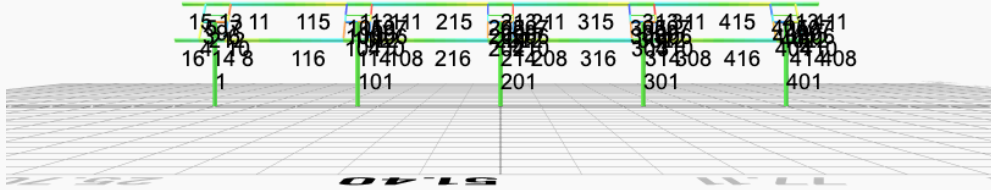
Top Bending Stress Z (ksi)



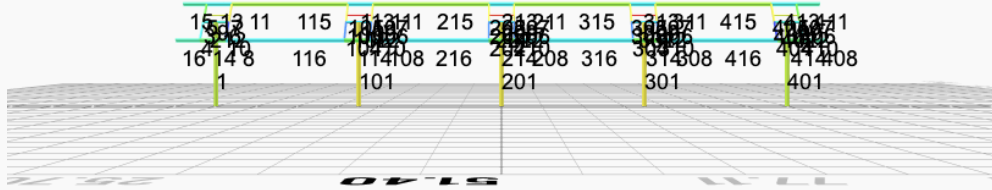
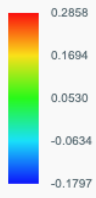
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 2. D + L	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 3. D + (S or Lr or R)	0.0762	4.2333	0.2178	0.8666	-0.3555	-0.9486
ULS: 3. D + (S or Lr or R)	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0677	3.8521	0.1937	0.7706	-0.3161	-0.8416
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 5b. D + 0.7E	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0677	3.8521	0.1937	0.7706	-0.3161	-0.8416
ULS: 8. 0.6D + 0.7E	0.0255	1.6250	0.0728	0.2896	-0.1188	-0.3123
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2593	5.4321	0.4555	1.7323	-2.2978	44.1318
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3390	-0.0115	-0.2050	-0.7363	1.8562	-43.9736
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4086	5.8948	0.4443	1.7078	-1.8910	32.6477
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0677	3.8521	0.1937	0.7706	-0.3161	-0.8416
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5402	1.8121	-0.0511	-0.1436	1.2245	-33.4314
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0677	3.8521	0.1937	0.7706	-0.3161	-0.8416
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4339	4.7511	0.3719	1.4199	-1.7729	32.9687
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5149	0.6684	-0.1234	-0.4315	1.3427	-33.1104
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0424	2.7084	0.1214	0.4827	-0.1980	-0.5205
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2763	4.3487	0.4069	1.5392	-2.2187	44.3400
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0255	1.6250	0.0728	0.2896	-0.1188	-0.3123
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3220	-1.0949	-0.2536	-0.9293	1.9354	-43.7654
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0255	1.6250	0.0728	0.2896	-0.1188	-0.3123

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	8.5535
Shear X	-5.5649
Shear Z	0.7558
Moment X	2.8739
Moment Y (Twist)	3.8427
Moment Z	74.5590

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

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

Result	Value (kip, kip-ft)
Axial	5.8948
Shear X	-3.3390
Shear Z	0.4555
Moment X	1.7323
Moment Y (Twist)	2.2978
Moment Z	44.3400

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	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 2. D + L	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 3. D + (S or Lr or R)	-0.0758	5.5132	-0.0143	-0.0605	0.1019	0.9649
ULS: 3. D + (S or Lr or R)	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0674	4.9904	-0.0127	-0.0538	0.0906	0.8600

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 5b. D + 0.7E	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0674	4.9904	-0.0127	-0.0538	0.0906	0.8600
ULS: 8. 0.6D + 0.7E	-0.0253	2.0531	-0.0048	-0.0202	0.0341	0.3272
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5620	7.2597	0.0070	0.0106	-0.1229	61.2028
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4842	-0.4214	-0.0187	-0.0630	0.2078	-58.1271
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4572	7.8688	-0.0015	-0.0205	-0.0441	46.3530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0674	4.9904	-0.0127	-0.0538	0.0906	0.8600
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3274	2.1080	-0.0207	-0.0757	0.2039	-43.1443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0674	4.9904	-0.0127	-0.0538	0.0906	0.8600
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4320	6.3002	0.0032	-0.0005	-0.0780	46.0384
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3526	0.5394	-0.0160	-0.0557	0.1700	-43.4589
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0422	3.4218	-0.0079	-0.0337	0.0568	0.5454
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5451	5.8910	0.0101	0.0241	-0.1456	60.9846
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0253	2.0531	-0.0048	-0.0202	0.0341	0.3272
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.5010	-1.7901	-0.0155	-0.0495	0.1851	-58.3452
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0253	2.0531	-0.0048	-0.0202	0.0341	0.3272

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.5453
Shear X	-7.5955
Shear Z	-0.0330
Moment X	-0.1167
Moment Y (Twist)	0.3596
Moment Z	103.4990

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.8688
Shear X	-4.5620
Shear Z	-0.0207
Moment X	-0.0757
Moment Y (Twist)	0.2078
Moment Z	61.2028

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	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 2. D + L	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 3. D + (S or Lr or R)	-0.0008	5.4379	0.0000	-0.0001	0.0001	0.1117
ULS: 3. D + (S or Lr or R)	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0007	4.9234	0.0000	-0.0001	0.0001	0.1013
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 5b. D + 0.7E	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0007	4.9234	0.0000	-0.0001	0.0001	0.1013
ULS: 8. 0.6D + 0.7E	-0.0003	2.0279	0.0000	-0.0000	0.0000	0.0421
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5021	7.1600	0.0000	-0.0001	0.0001	61.0366
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4984	-0.3974	0.0000	-0.0001	0.0001	-58.7720
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701

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	-3.3770	7.7585	0.0000	-0.0001	0.0001	45.8262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0007	4.9234	0.0000	-0.0001	0.0001	0.1013
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3734	2.0905	0.0000	-0.0001	0.0001	-44.0303
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0007	4.9234	0.0000	-0.0001	0.0001	0.1013
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3767	6.2150	0.0000	-0.0001	0.0001	45.7950
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3737	0.5469	0.0000	-0.0001	0.0001	-44.0615
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0005	3.3798	0.0000	-0.0000	0.0001	0.0701
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5019	5.8081	0.0000	-0.0000	0.0000	61.0086
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0003	2.0279	0.0000	-0.0000	0.0000	0.0421
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4986	-1.7493	0.0000	-0.0000	0.0000	-58.8001
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0003	2.0279	0.0000	-0.0000	0.0000	0.0421

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.3883
Shear X	-7.5063
Shear Z	0.0000
Moment X	-0.0012
Moment Y (Twist)	0.0011
Moment Z	103.3546

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.7585
Shear X	-4.5021
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	61.0366

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	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 2. D + L	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 3. D + (S or Lr or R)	-0.0757	5.5132	0.0143	0.0603	-0.1016	0.9649
ULS: 3. D + (S or Lr or R)	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0674	4.9904	0.0127	0.0536	-0.0904	0.8600
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 5b. D + 0.7E	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0674	4.9904	0.0127	0.0536	-0.0904	0.8600
ULS: 8. 0.6D + 0.7E	-0.0253	2.0531	0.0048	0.0202	-0.0340	0.3272
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5620	7.2597	-0.0070	-0.0107	0.1230	61.2028
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4842	-0.4214	0.0187	0.0629	-0.2077	-58.1271
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4572	7.8688	0.0015	0.0204	0.0443	46.3530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0674	4.9904	0.0127	0.0536	-0.0904	0.8600
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3274	2.1080	0.0207	0.0756	-0.2037	-43.1443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0674	4.9904	0.0127	0.0536	-0.0904	0.8600
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4320	6.3002	-0.0032	0.0004	0.0781	46.0384
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3526	0.5394	0.0160	0.0556	-0.1699	-43.4589
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0422	3.4218	0.0079	0.0336	-0.0566	0.5454

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5451	5.8910	-0.0101	-0.0242	0.1456	60.9846
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0253	2.0531	0.0048	0.0202	-0.0340	0.3272
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.5010	-1.7901	0.0155	0.0494	-0.1850	-58.3452
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0253	2.0531	0.0048	0.0202	-0.0340	0.3272

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.5454
Shear X	-7.5955
Shear Z	0.0331
Moment X	0.1171
Moment Y (Twist)	0.3601
Moment Z	103.4992

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.8688
Shear X	-4.5620
Shear Z	0.0207
Moment X	0.0756
Moment Y (Twist)	0.2077
Moment Z	61.2028

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	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 2. D + L	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 3. D + (S or Lr or R)	0.0762	4.2333	-0.2178	-0.8670	0.3557	-0.9484
ULS: 3. D + (S or Lr or R)	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0677	3.8521	-0.1937	-0.7710	0.3163	-0.8414
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 5b. D + 0.7E	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0677	3.8521	-0.1937	-0.7710	0.3163	-0.8414
ULS: 8. 0.6D + 0.7E	0.0255	1.6250	-0.0728	-0.2898	0.1188	-0.3123
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2593	5.4321	-0.4555	-1.7326	2.2979	44.1319
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3390	-0.0116	0.2050	0.7360	-1.8561	-43.9735
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4086	5.8948	-0.4443	-1.7082	1.8912	32.6478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0677	3.8521	-0.1937	-0.7710	0.3163	-0.8414
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5401	1.8121	0.0511	0.1432	-1.2244	-33.4313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0677	3.8521	-0.1937	-0.7710	0.3163	-0.8414
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4339	4.7511	-0.3719	-1.4202	1.7730	32.9688
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5148	0.6684	0.1234	0.4313	-1.3426	-33.1103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0424	2.7084	-0.1214	-0.4830	0.1981	-0.5204
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2763	4.3487	-0.4069	-1.5394	2.2187	44.3401
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0255	1.6250	-0.0728	-0.2898	0.1188	-0.3123
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3220	-1.0949	0.2536	0.9292	-1.9354	-43.7654
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0255	1.6250	-0.0728	-0.2898	0.1188	-0.3123

Worst Case Reactions LRFD

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

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.5534
Shear X	-5.5650
Shear Z	-0.7558
Moment X	-2.8762
Moment Y (Twist)	3.8441
Moment Z	74.5605

Result	Value (kip, kip-ft)
Axial	5.8948
Shear X	-3.3390
Shear Z	-0.4555
Moment X	-1.7326
Moment Y (Twist)	2.2979
Moment Z	44.3401

Project Details

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

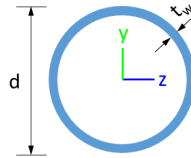


Design Input Information

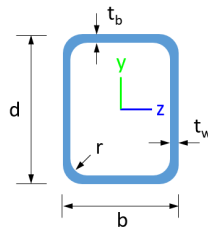
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

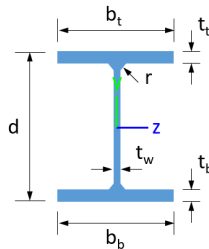
Section Dimensions



ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	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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315	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.17,1.14,1.17,1.15,1.17,1.14,1.17,1.15,1.17,1.09,1.17,1.15,1.17,1.13,1.17,1.15,1.17,1.14,1.17	300	200	1
316	19	8.42	8.42	12.95	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.17,1.19,1.17,1.19,1.17,1.19,1.18,1.19,1.13,1.19,1.18,1.19,1.16,1.19,1.17,1.19,1.17,1.19	300	200	1
401	10	27.86	27.86	13.27	-	300	200	1
402	5	1.30	1.30	2.00	-	300	200	1
403	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.19	300	200	1
404	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.64,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
405	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.67,1.68,1.67,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
406	16	0.92	0.92	1.42	1.18,1.17,1.18,1.17,1.18,1.18,1.16,1.17,1.16,1.17,1.16,1.18,1.16,1.18,1.17,1.17,1.07,1.17,1.17,1.18,1.15,1.18,1.16,1.18,1.16,1.18	300	200	1
407	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.60,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
408	19	4.20	4.20	2.00	2.33,2.33	300	200	1
409	2	2.60	2.60	4.00	-	300	200	1
410	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.59,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
411	19	4.20	4.20	2.00	2.33,2.33	300	200	1
412	5	1.30	1.30	2.00	-	300	200	1
413	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.16,1.10,1.21,1.10,1.16,1.10,1.19,1.10,1.13,1.10,1.45,1.10,1.15,1.10,1.24,1.10,1.17,1.10,1.19,1.10	300	200	1
414	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.08,1.10,1.08,1.10,1.08,1.10,1.07,1.10,1.09,1.10,1.23,1.10,1.08,1.10,1.09,1.10,1.08,1.10,1.07,1.10	300	200	1
415	19	8.42	8.42	12.95	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.11,1.10,1.10,1.10,1.11,1.10,1.10,1.10,1.25,1.10,1.10,1.10,1.11,2.1,1.10,1.10,1.10,1.10,1.10	300	200	1
416	19	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.10,1.11,1.09,1.11,1.10,1.11,1.10,1.11,1.10,1.11	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	574.32	213.67	123.94	123.94	172.30	172.30
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	123.95	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	25.28	6.12	40.24	43.62
14	133.20	85.85	24.59	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62
101	574.32	213.67	123.94	123.94	172.30	172.30
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28

104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.79	6.12	40.24	43.62
114	133.20	85.85	23.69	6.12	40.24	43.62
115	133.20	46.28	11.90	6.12	40.24	43.62
116	133.20	46.28	12.02	6.12	40.24	43.62
201	574.32	213.67	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.50	6.12	40.24	43.62
214	133.20	85.85	23.51	6.12	40.24	43.62
215	133.20	46.28	12.34	6.12	40.24	43.62
216	133.20	46.28	12.69	6.12	40.24	43.62
301	574.32	213.67	123.94	123.94	172.30	172.30
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.79	6.12	40.24	43.62
314	133.20	85.85	23.69	6.12	40.24	43.62
315	133.20	46.28	11.76	6.12	40.24	43.62
316	133.20	46.28	12.23	6.12	40.24	43.62
401	574.32	213.67	123.94	123.94	172.30	172.30
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28

407	110.10	114.23	13.79	11.10	42.00	23.20
408	133.20	102.39	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	102.39	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	25.28	6.12	40.24	43.62
414	133.20	85.85	24.58	6.12	40.24	43.62
415	133.20	46.28	11.93	6.12	40.24	43.62
416	133.20	46.28	11.76	6.12	40.24	43.62

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	0.040	0.602	0.058	0.032	0.004	0.645	#13	0.581	Not Required	Pass
2	0.003	0.128	0.149	0.040	0.035	0.278	#13	0.035	Not Required	Pass
3	0.005	0.378	0.037	0.035	0.011	0.381	#13	0.045	Not Required	Pass
4	0.005	0.382	0.090	0.038	0.022	0.472	#13	0.120	Not Required	Pass
5	0.004	0.235	0.057	0.038	0.016	0.241	#13	0.074	Not Required	Pass
6	0.012	0.730	0.104	0.076	0.026	0.808	#13	0.045	Not Required	Pass
7	0.012	0.453	0.217	0.072	0.053	0.484	#13	0.074	Not Required	Pass
8	0.007	0.136	0.240	0.046	0.019	0.246	#23	0.095	Not Required	Pass
9	0.011	0.079	0.102	0.006	0.006	0.168	#13	0.204	Not Required	Pass
10	0.013	0.688	0.213	0.069	0.045	0.705	#13	0.080	Not Required	Pass
11	0.007	0.118	0.248	0.049	0.019	0.257	#23	0.095	Not Required	Pass
12	0.003	0.452	0.324	0.092	0.063	0.777	#13	0.053	Not Required	Pass
13	0.010	0.133	0.514	0.061	0.023	0.568	#21	0.286	Not Required	Pass
14	0.007	0.110	0.504	0.058	0.023	0.528	#21	0.190	Not Required	Pass
15	0.000	0.017	0.036	0.014	0.005	0.048	#21	Not Required	Not Required	Pass
16	0.000	0.017	0.036	0.014	0.005	0.048	#21	Not Required	Not Required	Pass
101	0.054	0.835	0.003	0.044	0.000	0.862	#13	0.581	Not Required	Pass
102	0.005	0.400	0.334	0.091	0.062	0.736	#13	0.035	Not Required	Pass
103	0.011	0.738	0.057	0.073	0.002	0.781	#13	0.045	Not Required	Pass
104	0.011	0.771	0.220	0.077	0.045	0.873	#13	0.080	Not Required	Pass
105	0.011	0.458	0.232	0.073	0.059	0.500	#13	0.074	Not Required	Pass
106	0.011	0.763	0.057	0.076	0.004	0.797	#13	0.045	Not Required	Pass
107	0.011	0.475	0.221	0.076	0.057	0.516	#13	0.074	Not Required	Pass
108	0.008	0.077	0.228	0.051	0.019	0.286	#21	0.095	Not Required	Pass
109	0.023	0.053	0.067	0.001	0.000	0.127	#13	0.204	Not Required	Pass
110	0.011	0.767	0.209	0.077	0.043	0.853	#13	0.080	Not Required	Pass
111	0.007	0.055	0.237	0.050	0.019	0.282	#21	0.095	Not Required	Pass
112	0.005	0.408	0.344	0.091	0.065	0.754	#13	0.035	Not Required	Pass
113	0.010	0.250	0.526	0.065	0.024	0.691	#21	0.286	Not Required	Pass
114	0.012	0.301	0.519	0.069	0.024	0.717	#21	0.286	Not Required	Pass
115	0.018	0.617	0.274	0.053	0.019	0.798	#13	0.601	Not Required	Pass
116	0.009	0.591	0.270	0.057	0.019	0.774	#13	0.601	Not Required	Pass
201	0.053	0.834	0.000	0.044	0.000	0.861	#13	0.581	Not Required	Pass
202	0.005	0.400	0.336	0.090	0.063	0.738	#13	0.035	Not Required	Pass
203	0.011	0.758	0.055	0.075	0.002	0.798	#13	0.045	Not Required	Pass
204	0.011	0.742	0.207	0.074	0.043	0.831	#13	0.080	Not Required	Pass
205	0.011	0.471	0.219	0.075	0.056	0.509	#13	0.074	Not Required	Pass

206	0.011	0.758	0.055	0.075	0.002	0.798	#13	0.045	Not Required	Pass
207	0.011	0.471	0.219	0.075	0.056	0.509	#13	0.074	Not Required	Pass
208	0.008	0.075	0.227	0.051	0.019	0.282	#21	0.095	Not Required	Pass
209	0.022	0.048	0.069	0.001	0.000	0.124	#13	0.204	Not Required	Pass
210	0.011	0.742	0.207	0.074	0.043	0.831	#13	0.080	Not Required	Pass
211	0.007	0.076	0.234	0.052	0.019	0.293	#21	0.095	Not Required	Pass
212	0.005	0.400	0.336	0.090	0.063	0.738	#13	0.035	Not Required	Pass
213	0.010	0.299	0.497	0.064	0.023	0.706	#21	0.286	Not Required	Pass
214	0.012	0.300	0.488	0.063	0.023	0.688	#21	0.286	Not Required	Pass
215	0.019	0.439	0.274	0.052	0.019	0.617	#13	0.601	Not Required	Pass
216	0.012	0.409	0.269	0.051	0.019	0.590	#13	0.601	Not Required	Pass
301	0.054	0.835	0.003	0.044	0.000	0.862	#13	0.581	Not Required	Pass
302	0.005	0.408	0.344	0.091	0.065	0.754	#13	0.035	Not Required	Pass
303	0.011	0.763	0.057	0.076	0.004	0.797	#13	0.045	Not Required	Pass
304	0.011	0.767	0.209	0.077	0.043	0.853	#13	0.080	Not Required	Pass
305	0.011	0.475	0.221	0.076	0.057	0.516	#13	0.074	Not Required	Pass
306	0.011	0.738	0.057	0.073	0.002	0.781	#13	0.045	Not Required	Pass
307	0.011	0.458	0.232	0.073	0.059	0.500	#13	0.074	Not Required	Pass
308	0.007	0.064	0.252	0.057	0.019	0.299	#21	0.095	Not Required	Pass
309	0.023	0.053	0.067	0.001	0.000	0.127	#13	0.204	Not Required	Pass
310	0.011	0.771	0.220	0.077	0.045	0.873	#13	0.080	Not Required	Pass
311	0.007	0.092	0.259	0.053	0.019	0.287	#21	0.095	Not Required	Pass
312	0.005	0.400	0.334	0.091	0.062	0.736	#13	0.035	Not Required	Pass
313	0.010	0.250	0.526	0.065	0.024	0.692	#21	0.286	Not Required	Pass
314	0.012	0.301	0.519	0.069	0.024	0.717	#21	0.286	Not Required	Pass
315	0.019	0.445	0.274	0.050	0.019	0.623	#13	0.601	Not Required	Pass
316	0.012	0.408	0.269	0.051	0.019	0.589	#13	0.601	Not Required	Pass
401	0.040	0.602	0.058	0.032	0.004	0.645	#13	0.581	Not Required	Pass
402	0.003	0.452	0.324	0.092	0.063	0.777	#13	0.053	Not Required	Pass
403	0.012	0.730	0.104	0.076	0.026	0.808	#13	0.045	Not Required	Pass
404	0.013	0.688	0.213	0.069	0.045	0.705	#13	0.080	Not Required	Pass
405	0.012	0.453	0.217	0.072	0.053	0.484	#13	0.074	Not Required	Pass
406	0.005	0.378	0.037	0.035	0.011	0.381	#13	0.045	Not Required	Pass
407	0.004	0.235	0.057	0.038	0.016	0.241	#13	0.074	Not Required	Pass
408	0.000	0.017	0.036	0.014	0.005	0.048	#21	Not Required	Not Required	Pass
409	0.011	0.079	0.102	0.006	0.006	0.168	#13	0.204	Not Required	Pass
410	0.005	0.382	0.090	0.038	0.022	0.472	#13	0.120	Not Required	Pass
411	0.000	0.017	0.036	0.014	0.005	0.048	#21	Not Required	Not Required	Pass
412	0.003	0.128	0.149	0.040	0.035	0.278	#13	0.035	Not Required	Pass
413	0.010	0.133	0.513	0.061	0.023	0.568	#21	0.190	Not Required	Pass
414	0.007	0.110	0.504	0.058	0.023	0.528	#21	0.286	Not Required	Pass
415	0.018	0.627	0.275	0.049	0.019	0.807	#13	0.601	Not Required	Pass
416	0.009	0.624	0.267	0.046	0.019	0.805	#13	0.601	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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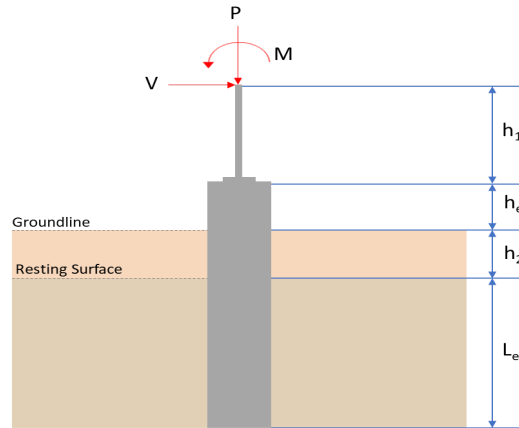
SkyCiv Foundation Design

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

$L = 7.5$ 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)	5.895	8.554
V_x (kip)	-3.339	-5.565
V_z (kip)	0.455	0.756
M_x (kipft)	1.732	2.874
M_z (kipft)	44.340	74.559

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,

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

$$H_o = \frac{(-3.339 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.53169 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.34 \text{ kipft}) + ((-3.339 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0605 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.9918 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.455 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.072452 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.732 \text{ kipft}) + ((0.455 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.2758 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.3166 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9918 \text{ ft}), (3.3166 \text{ ft})]$$

$$L_{e,req} = 6.992 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.992 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93227$$

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

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

$$q = \frac{(5.895 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36844 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36844 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18422$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.53169 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.0605 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0605 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.0605 \text{ kipft/ft})) + (4 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.171 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.0605 \text{ kipft/ft})) + (3 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.0605 \text{ kipft/ft})) + (2 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.26756 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.0605 \text{ kipft/ft})) + ((-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0809 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.171 \text{ ft})}{2}$$

$$p_a = 0.38782 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26756 \text{ kip/ft}^2)}{(0.38782 \text{ kip/ft}^2)}$$

$$Ratio = 0.6899$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0809 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.96079$$

Status: **PASS**
Ratio: **0.690**

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

Considering z-direction:

$H_o = 0.072452 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.2758 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.2758 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.072452 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.2758 \text{ kipft/ft})) + (4 \times (0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3548 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.2758 \text{ kipft/ft})) + (3 \times (0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.2758 \text{ kipft/ft})) + (2 \times (0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.052042 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.2758 \text{ kipft/ft})) + ((0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.1168 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3548 \text{ ft})}{2}$$

$$p_a = 0.40161 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.052042 \text{ kip/ft}^2)}{(0.40161 \text{ kip/ft}^2)}$$

$$Ratio = 0.12958$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

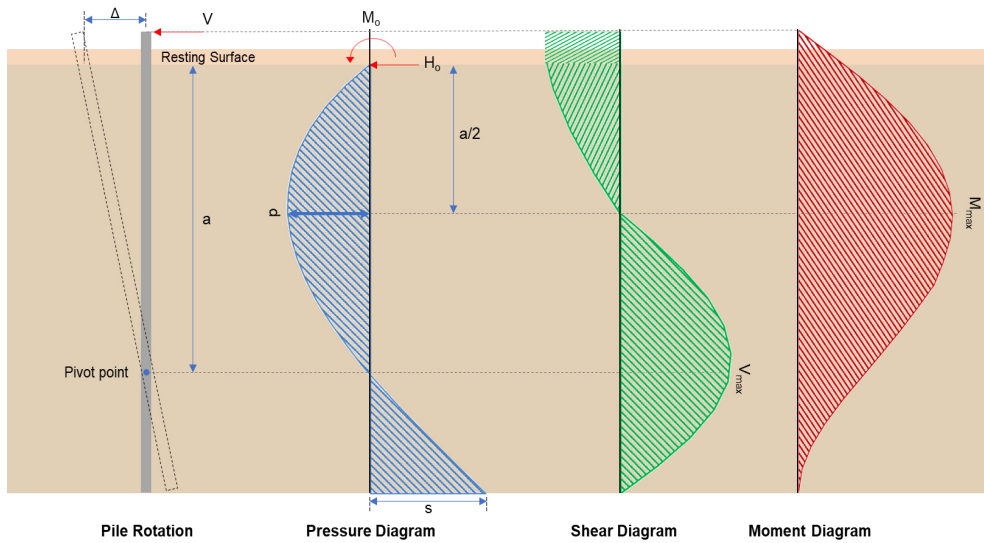
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.1168 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.10382$$

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

Status: **PASS**
Ratio: **0.100**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.565 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.88615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(74.559 \text{ kipft}) + ((-5.565 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.872 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.872 \text{ kipft/ft})}{(-0.88615 \text{ kip/ft})}$$

$$E = 13.398 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.872 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.872 \text{ kipft/ft})) + (4 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.88615 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.872 \text{ kipft/ft})) + (4 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1699 \text{ ft}$$

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

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

$$V_{max} = ((-0.88615 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1699 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1699 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.543 \text{ kip}$$

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

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

$$M_{max} = ((-0.88615 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(13.398 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1699 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1699 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1699 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 48.371 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.756 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.12038 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.874 \text{ kipft}) + ((0.756 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.45764 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.45764 \text{ kipft/ft})}{(0.12038 \text{ kip/ft})}$$

$$E = 3.8016 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.45764 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.12038 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.45764 \text{ kipft/ft})) + (4 \times (0.12038 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.355 \text{ ft}$$

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

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

$$V_{max} = ((0.12038 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8016 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.355 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.8016 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.355 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.75266 \text{ kip}$$

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

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

$$M_{max} = ((0.12038 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.8016 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.355 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.8016 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.355 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.8016 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.355 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.5003 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.554 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.312 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.312 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.554 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0031975$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.554 \text{ kip} \rightarrow 8554 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8554 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.63 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.63 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.63 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.63 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.84 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.543 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.543 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.12219$$

Considering z-direction:

$V_{max} = 0.75266 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.75266 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.0067906$$

Status: **PASS**
Ratio: **0.120**

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

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:

$M_{max} = 48.371 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(48.371 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19379$$

Status: **PASS**
Ratio: **0.190**

Considering z-direction:

$M_{max} = 2.5003 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(2.5003 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010017$$

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

REFERENCES	CALCULATIONS	RESULTS
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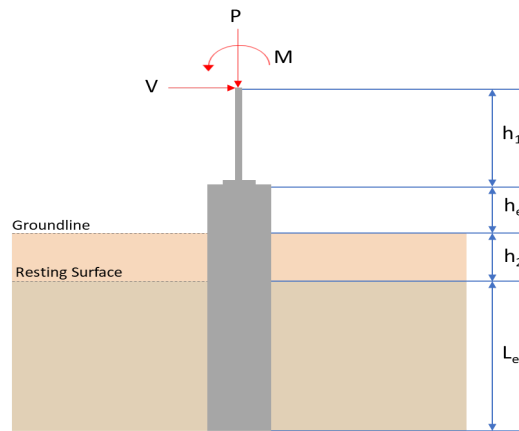
SkyCiv Foundation Design

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

$L = 7.5$ 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)	5.895	8.553
V_x (kip)	-3.339	-5.565
V_z (kip)	-0.455	-0.756
M_x (kipft)	-1.733	-2.876
M_z (kipft)	44.340	74.560

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,

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

$$H_o = \frac{(-3.339 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.53169 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.34 \text{ kipft}) + ((-3.339 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0605 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.9918 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.455 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.072452 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.733 \text{ kipft}) + ((-0.455 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.27596 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.2956 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9918 \text{ ft}), (2.2956 \text{ ft})]$$

$$L_{e,req} = 6.992 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.992 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93227$$

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

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

$$q = \frac{(5.895 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36844 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36844 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18422$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.53169 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.0605 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0605 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.0605 \text{ kipft/ft})) + (4 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.171 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.0605 \text{ kipft/ft})) + (3 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.0605 \text{ kipft/ft})) + (2 \times (-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.26756 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.0605 \text{ kipft/ft})) + ((-0.53169 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0809 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.171 \text{ ft})}{2}$$

$$p_a = 0.38782 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26756 \text{ kip/ft}^2)}{(0.38782 \text{ kip/ft}^2)}$$

$$Ratio = 0.6899$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0809 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.96079$$

Status: **PASS**
Ratio: **0.690**

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

Considering z-direction:

$H_o = -0.072452 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.27596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.27596 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.072452 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.27596 \text{ kipft/ft})) + (4 \times (-0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3548 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.27596 \text{ kipft/ft})) + (3 \times (-0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.27596 \text{ kipft/ft})) + (2 \times (-0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.014267 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.27596 \text{ kipft/ft})) + ((-0.072452 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.0009087 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3548 \text{ ft})}{2}$$

$$p_a = 0.40161 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.014267 \text{ kip/ft}^2)}{(0.40161 \text{ kip/ft}^2)}$$

$$Ratio = -0.035525$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

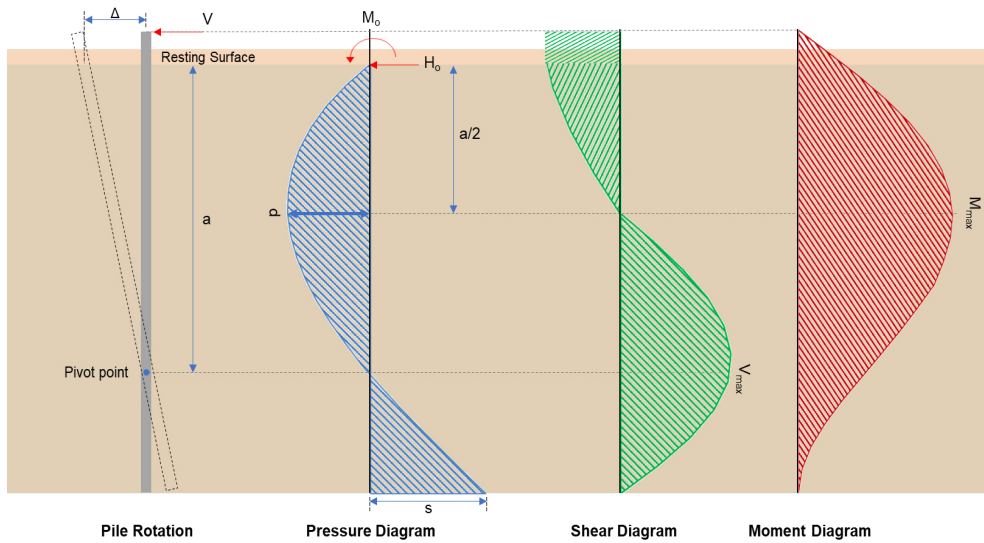
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0009087 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.00080774$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.565 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.88615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(74.56 \text{ kipft}) + ((-5.565 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.873 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.873 \text{ kipft/ft})}{(-0.88615 \text{ kip/ft})}$$

$$E = 13.398 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.873 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.873 \text{ kipft/ft})) + (4 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(6 \times (11.873 \text{ kipft/ft})) + (4 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft}))}{(6 \times (11.873 \text{ kipft/ft})) + (4 \times (-0.88615 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1699 \text{ ft}$$

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

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

$$V_{max} = ((-0.88615 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1699 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1699 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.543 \text{ kip}$$

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

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

$$M_{max} = ((-0.88615 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(13.398 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1699 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1699 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (13.398 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1699 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 48.371 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.756 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.12038 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.876 \text{ kipft}) + ((-0.756 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.45796 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.45796 \text{ kipft/ft})}{(-0.12038 \text{ kip/ft})}$$

$$E = 3.8042 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.45796 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.12038 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.45796 \text{ kipft/ft})) + (4 \times (-0.12038 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3549 \text{ ft}$$

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

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

$$V_{max} = ((-0.12038 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8042 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3549 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.8042 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3549 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.75295 \text{ kip}$$

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

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

$$M_{max} = ((-0.12038 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.8042 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3549 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.8042 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3549 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (3.8042 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3549 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 2.5014 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.553 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.312 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.312 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.553 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0031972$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.553 \text{ kip} \rightarrow 8553 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8553 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.63 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.63 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.63 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.63 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.84 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.543 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(13.543 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.12219$$

Considering z-direction:

$V_{max} = 0.75295 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.75295 \text{ kip})}{(110.84 \text{ kip})}$$

$$Ratio = 0.0067933$$

Status: **PASS**
Ratio: **0.120**

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

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:

$M_{max} = 48.371 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(48.371 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19379$$

Status: **PASS**
Ratio: **0.190**

Considering z-direction:

$M_{max} = 2.5014 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.5014 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010022$$

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

REFERENCES	CALCULATIONS	RESULTS
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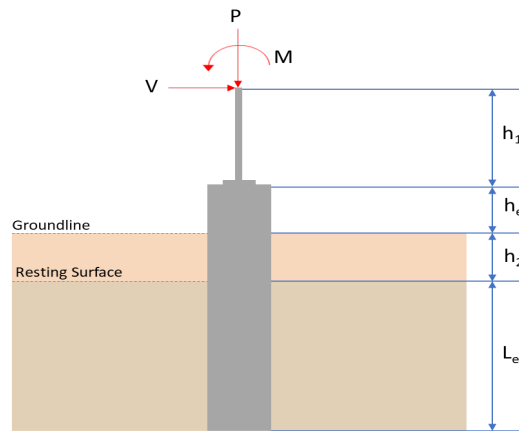
SkyCiv Foundation Design

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

$L = 8.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.869	11.545
V_x (kip)	-4.562	-7.595
V_z (kip)	-0.021	-0.033
M_x (kipft)	-0.076	-0.117
M_z (kipft)	61.203	103.499

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,

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

$$H_o = \frac{(-4.562 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.72643 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.203 \text{ kipft}) + ((-4.562 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.7457 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.6433 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.021 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0033439 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.076 \text{ kipft}) + ((-0.021 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.012102 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.9218 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.6433 \text{ ft}), (0.9218 \text{ ft})]$$

$$L_{e,req} = 7.643 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.643 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.92642$$

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

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

$$q = \frac{(7.869 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49181 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49181 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24591$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.72643 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.7457 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7457 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.7457 \text{ kipft/ft})) + (4 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.7457 \text{ kipft/ft})) + (3 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.7457 \text{ kipft/ft})) + (2 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.28179 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.7457 \text{ kipft/ft})) + ((-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1899 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7 \text{ ft})}{2}$$

$$p_a = 0.42749 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.28179 \text{ kip/ft}^2)}{(0.42749 \text{ kip/ft}^2)}$$

$$Ratio = 0.65917$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1899 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.96156$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = -0.0033439 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.012102 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012102 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.012102 \text{ kipft/ft})) + (4 \times (-0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9147 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.012102 \text{ kipft/ft})) + (3 \times (-0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 [(3 \times (0.012102 \text{ kipft/ft})) + (2 \times (-0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = -0.00068925 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.012102 \text{ kipft/ft})) + ((-0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = -0.00029829 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9147 \text{ ft})}{2}$$

$$p_a = 0.4436 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00068925 \text{ kip/ft}^2)}{(0.4436 \text{ kip/ft}^2)}$$

$$Ratio = -0.0015538$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

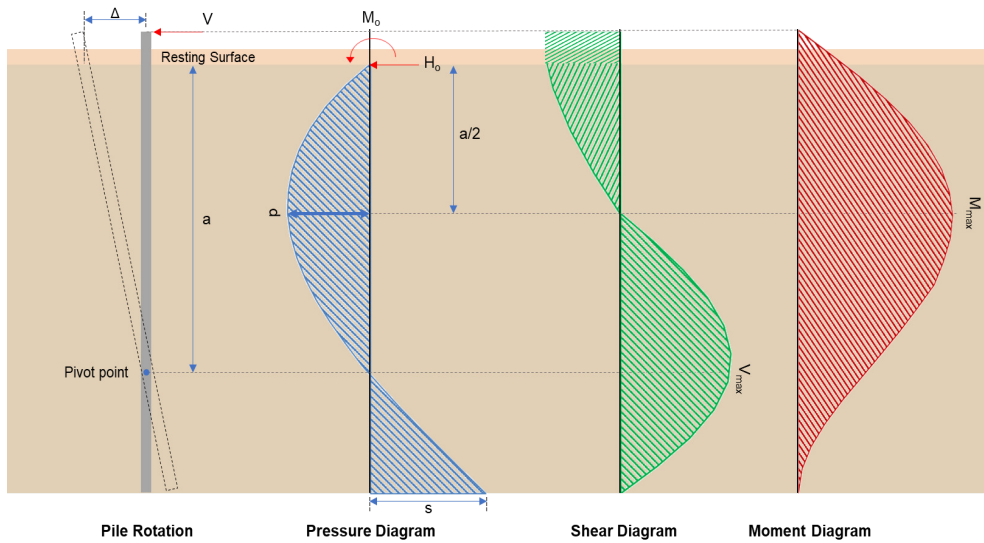
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00029829 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = -0.00024104$$

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,

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

$$H_o = \frac{(-7.595 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2094 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.5 \text{ kipft}) + ((-7.595 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.481 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.481 \text{ kipft/ft})}{(-1.2094 \text{ kip/ft})}$$

$$E = 13.627 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.481 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.2094 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 16.481 \text{ kipft/ft}) + (4 \times (-1.2094 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (16.481 \text{ kipft/ft})) + (4 \times (-1.2094 \text{ kip/ft}) \times (8.25 \text{ ft}))}{(6 \times (16.481 \text{ kipft/ft})) + (4 \times (-1.2094 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6977 \text{ ft}$$

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

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

$$V_{max} = ((-1.2094 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6977 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6977 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.33 \text{ kip}$$

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

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

$$M_{max} = ((-1.2094 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(13.627 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.864 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.033 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0052548 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.117 \text{ kipft}) + ((-0.033 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018631 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018631 \text{ kipft/ft})}{(-0.0052548 \text{ kip/ft})}$$

$$E = 3.5455 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018631 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0052548 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.018631 \text{ kipft/ft})) + (4 \times (-0.0052548 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.918 \text{ ft}$$

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

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

$$V_{max} = ((-0.0052548 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.918 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.918 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.030021 \text{ kip}$$

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

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

$$M_{max} = ((-0.0052548 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(3.5455 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1084 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.545 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.212 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.212 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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;">$Ratio = \frac{(11.545 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0043156$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.545 \text{ kip} \rightarrow 11545 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11545 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.02 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.02 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.02 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.02 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.1 \text{ kip}$$

Considering x-direction:

V_{max} = 17.33 kip - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(17.33 \text{ kip})}{(111.1 \text{ kip})}$$

$$Ratio = 0.15599$$

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

Considering z-direction:

$V_{max} = 0.030021 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.030021 \text{ kip})}{(111.1 \text{ kip})}$$

$$Ratio = 0.00027022$$

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:

$M_{max} = 67.864 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.864 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27189$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.1084 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.1084 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0004343$$

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

REFERENCES	CALCULATIONS	RESULTS
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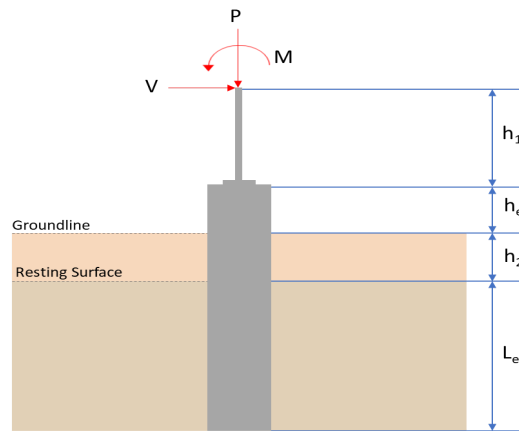
SkyCiv Foundation Design

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

$L = 8.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.759	11.388
V_x (kip)	-4.502	-7.506
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	61.037	103.355

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,

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

$$H_o = \frac{(-4.502 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.71688 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.037 \text{ kipft}) + ((-4.502 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.7193 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.6536 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(7.6536 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.654 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.654 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.92776$$

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

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

$$q = \frac{(7.759 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.48494 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.48494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24247$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.71688 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.7193 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7193 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.71688 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.7193 \text{ kipft/ft})) + (4 \times (-0.71688 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6984 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (9.7193 \text{ kipft/ft})) + (3 \times (-0.71688 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 [(3 \times (9.7193 \text{ kipft/ft})) + (2 \times (-0.71688 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.28402 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (9.7193 \text{ kipft/ft})) + ((-0.71688 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1922 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6984 \text{ ft})}{2}$$

$$p_a = 0.42738 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28402 \text{ kip/ft}^2)}{(0.42738 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66456$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

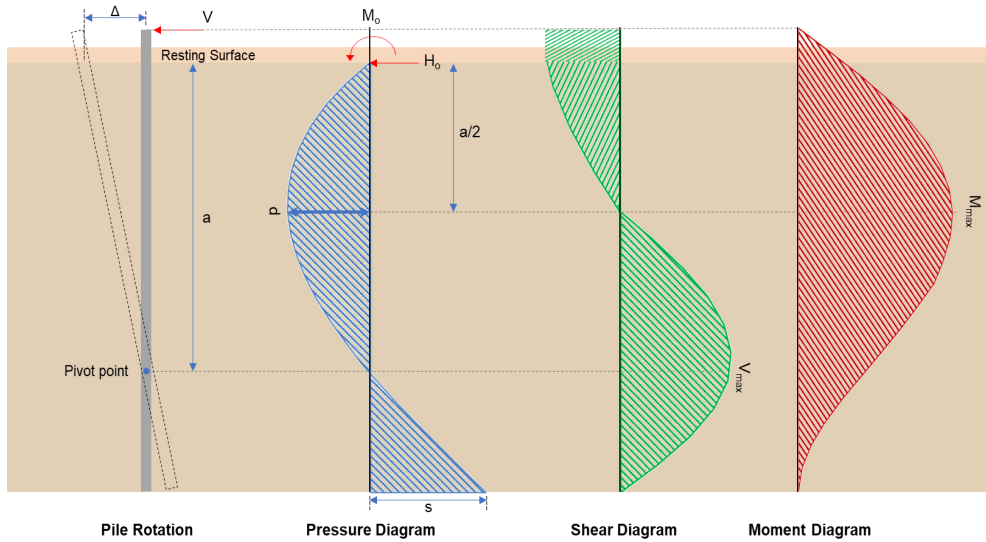
$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1922 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.660**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.506 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1952 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.36 \text{ kipft}) + ((-7.506 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.458 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.458 \text{ kipft/ft})}{(-1.1952 \text{ kip/ft})}$$

$$E = 13.77 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.458 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.1952 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (16.458 \text{ kipft/ft})) + (4 \times (-1.1952 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6962 \text{ ft}$$

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

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

$$V_{max} = ((-1.1952 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.77 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6962 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.77 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6962 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.213 \text{ kip}$$

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

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

$$M_{max} = ((-1.1952 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(13.77 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6962 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.77 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6962 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.77 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6962 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.671 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00015924 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.5 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(8.25 \text{ ft})} \right) \times \left(\frac{(5.5 \text{ ft})}{(8.25 \text{ ft})} - 1 \right) \times \left(\frac{(5.5 \text{ ft})}{(8.25 \text{ ft})} \right)^2$$

$$V_{max} = 0.00013725 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(5.5 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left(3 \times \frac{(5.5 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\left(\frac{11.388 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.218 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.218 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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,

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

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

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

Summary:

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

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

22.4.2.2 **Axial Compression Strength (ACI 318-19, LRFD)**

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

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

$$Ratio = \frac{(11.388 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0042569$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

$b_w = 48$ 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.388 \text{ kip} \rightarrow 11388 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11388 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

$$V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = MIN [(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 17.273 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.273 \text{ kip})}{(111.08 \text{ kip})}$$

$$\text{Ratio} = 0.15549$$

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

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 $\phi M_{n,2}$

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

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

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

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:

$M_{max} = 67.671 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(67.671 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.27112$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 2.2683 \times 10^{-6}$$

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

REFERENCES	CALCULATIONS	RESULTS
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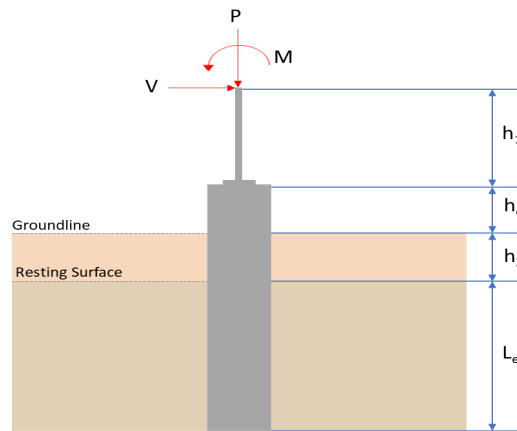
SkyCiv Foundation Design

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

$L = 8.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.869	11.545
V_x (kip)	-4.562	-7.595
V_z (kip)	0.021	0.033
M_x (kipft)	0.076	0.117
M_z (kipft)	61.203	103.499

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,

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

$$H_o = \frac{(-4.562 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.72643 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.203 \text{ kipft}) + ((-4.562 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.7457 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.6433 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.021 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0033439 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.076 \text{ kipft}) + ((0.021 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.012102 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0566 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.6433 \text{ ft}), (1.0566 \text{ ft})]$$

$$L_{e,req} = 7.643 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.643 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.92642$$

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

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

$$q = \frac{(7.869 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49181 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49181 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24591$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.72643 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.7457 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7457 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.7457 \text{ kipft/ft})) + (4 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.7457 \text{ kipft/ft})) + (3 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.7457 \text{ kipft/ft})) + (2 \times (-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.28179 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.7457 \text{ kipft/ft})) + ((-0.72643 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1899 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7 \text{ ft})}{2}$$

$$p_a = 0.42749 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.28179 \text{ kip/ft}^2)}{(0.42749 \text{ kip/ft}^2)}$$

$$Ratio = 0.65917$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1899 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.96156$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = 0.0033439 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.012102 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012102 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.012102 \text{ kipft/ft})) + (4 \times (0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9147 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.012102 \text{ kipft/ft})) + (3 \times (0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.012102 \text{ kipft/ft})) + (2 \times (0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.0020725 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.012102 \text{ kipft/ft})) + ((0.0033439 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.0045656 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9147 \text{ ft})}{2}$$

$$p_a = 0.4436 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0020725 \text{ kip/ft}^2)}{(0.4436 \text{ kip/ft}^2)}$$

$$Ratio = 0.004672$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

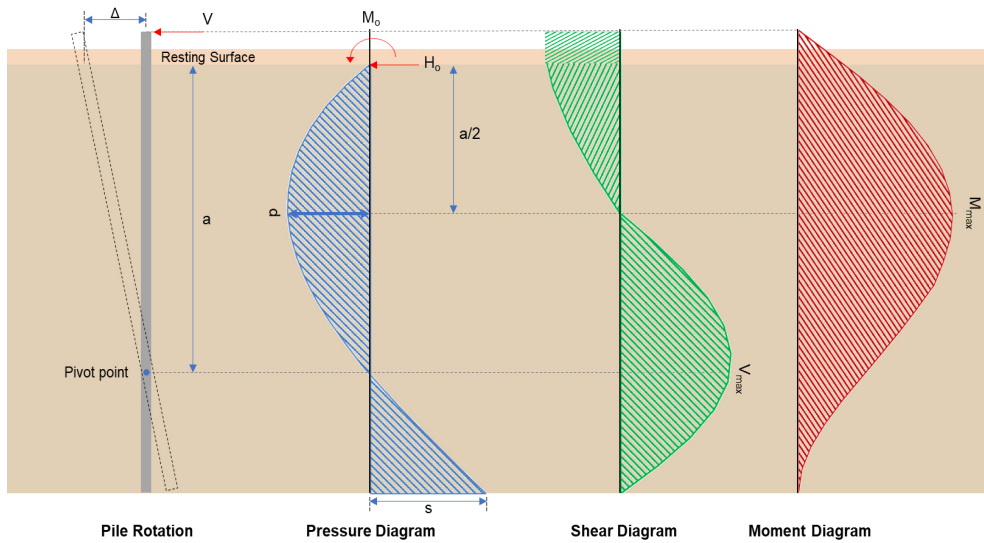
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0045656 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0036894$$

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,

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

$$H_o = \frac{(-7.595 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2094 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.5 \text{ kipft}) + ((-7.595 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.481 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.481 \text{ kipft/ft})}{(-1.2094 \text{ kip/ft})}$$

$$E = 13.627 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.481 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.2094 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 16.481 \text{ kipft/ft}) + (4 \times (-1.2094 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (16.481 \text{ kipft/ft})) + (4 \times (-1.2094 \text{ kip/ft}) \times (8.25 \text{ ft}))}{}$$

$$a = 5.6977 \text{ ft}$$

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

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

$$V_{max} = ((-1.2094 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6977 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6977 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.33 \text{ kip}$$

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

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

$$M_{max} = ((-1.2094 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(13.627 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.627 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6977 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.864 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.033 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0052548 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.117 \text{ kipft}) + ((0.033 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018631 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018631 \text{ kipft/ft})}{(0.0052548 \text{ kip/ft})}$$

$$E = 3.5455 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018631 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0052548 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.018631 \text{ kipft/ft})) + (4 \times (0.0052548 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.918 \text{ ft}$$

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

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

$$V_{max} = ((0.0052548 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.918 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.918 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.030021 \text{ kip}$$

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

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

$$M_{max} = ((0.0052548 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(3.5455 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5455 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.918 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1084 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.545 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.212 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.212 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(11.545 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0043156$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.545 \text{ kip} \rightarrow 11545 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11545 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.02 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.02 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.02 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.02 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.1 \text{ kip}$$

Considering x-direction:

V_{max} = 17.33 kip - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(17.33 \text{ kip})}{(111.1 \text{ kip})}$$

$$Ratio = 0.15599$$

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

Considering z-direction:

$V_{max} = 0.030021 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.030021 \text{ kip})}{(111.1 \text{ kip})}$$

$$Ratio = 0.00027022$$

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:

$M_{max} = 67.864 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.864 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27189$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.1084 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.1084 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0004343$$

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