

Project Name: MTSOLAR_7AJLJ4L53AC4B **Date:** Thu Feb 13 2025
Location: 4675 KY-181, Greenville, KY 42345, USA **Number of Modules:** 45
Unique ID: 4P-22.5-6TOP-HD-57-L-3Hx15W-KHH4 **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	11.38 ft
Array Dimensions E/W	86.25 ft
Winter Tilt Angle	30
Front Edge Clearance	5 ft

MT Solar Bill of Materials (4P-22.5-6TOP-HD-57-L-3Hx15W-KHH4)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	12
MTS-CLAMP-HOOK-4PK	Hook Clamp	15

Rail Bill of Materials

Part	Qty
Rails (135in)	30
Rail Attachment	60
Module Mid Clamp	60
Module End Clamp	60
Ground Lug	15

Site Details:



Site Address: 4675 KY-181, Greenville, KY 42345, USA

Array Specification

Duty Classification:	HD
Module Width:	45.00 in
Module Length:	68.00in
Number of Rows:	3
Number of Columns:	15
Total Number of Modules:	45
Winter Tilt Angle:	30
Front Edge Clearance:	5
Total Array Height at Tilt:	10.69 ft
Total Frame Length:	84.50 ft
Frame Weight:	4106 lbs
Array Dimensions N/S:	11.38 ft
Array Dimensions E/W:	86.25 ft
Rail Length:	136.50 in
Rail Spacing:	2.88 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	7.84 ft
Number of Poles:	4
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.00 ft
Foundation Volume:	6.414 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sedimentary
Site Location:	4675 KY-181, Greenville, KY 42345, USA
Wind Speed:	99 mph

Snow Load:

15 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.

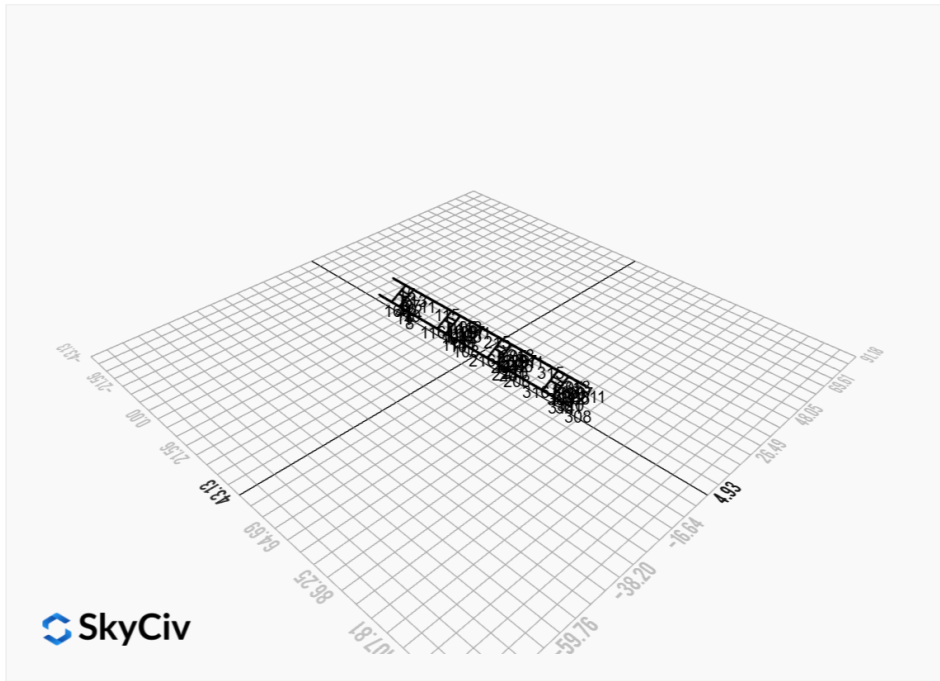
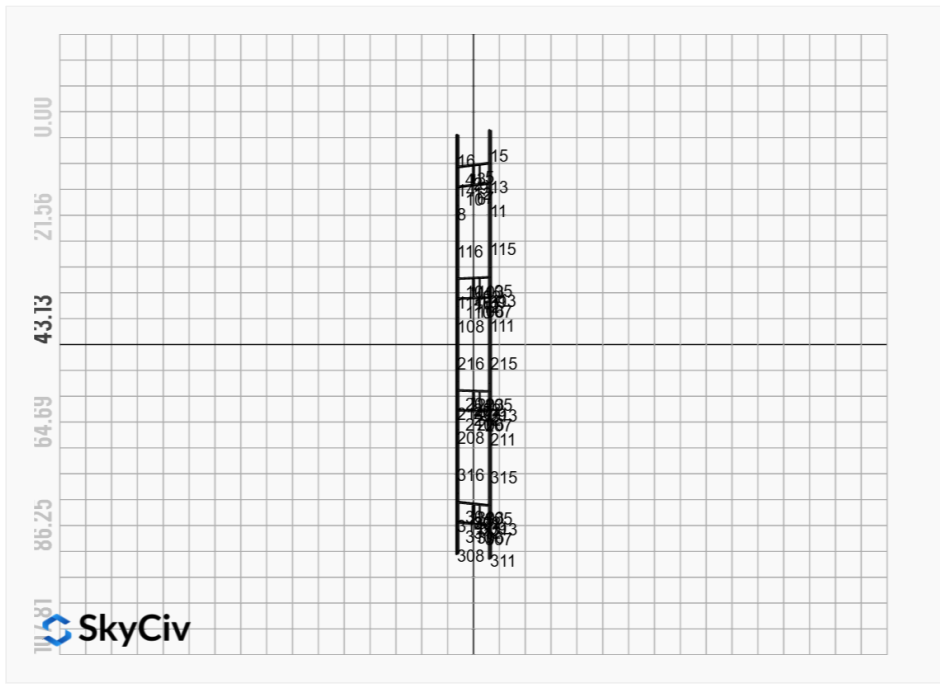
AutoDesigner Input

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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)

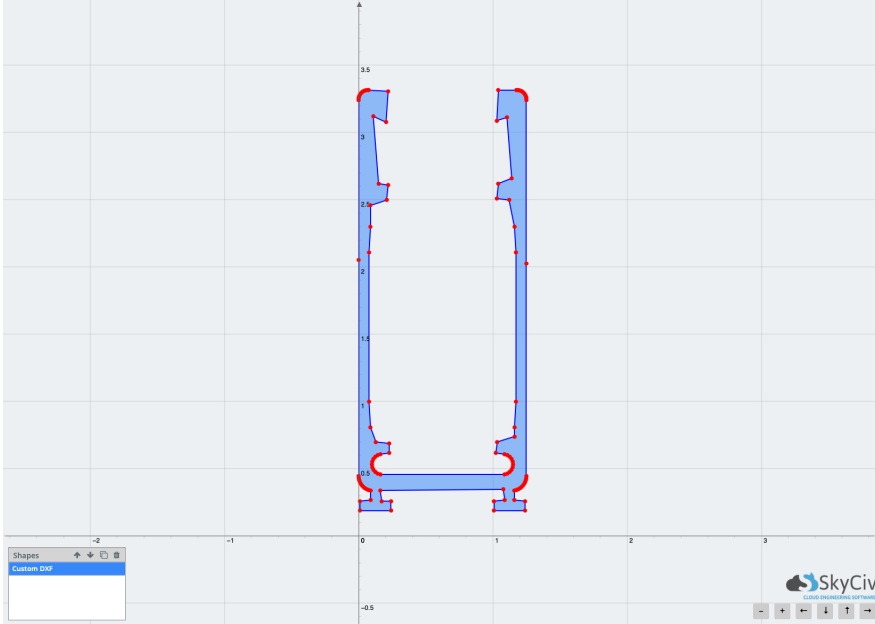






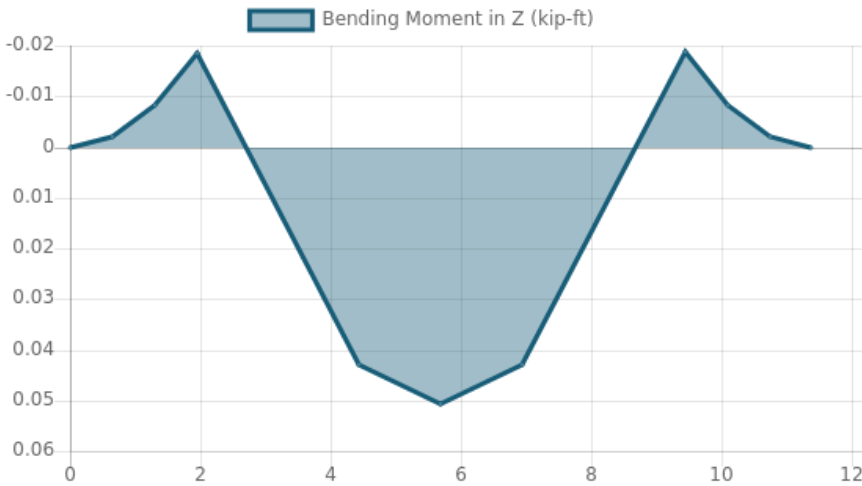
Rail Design Check

Rail Length: 11.375 ft
Additional Restraints Required: None
Tributary Width: 2.875 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0164 kip/ft
Snow (Y): -0.0095 kip/ft
Wind uplift Case A: 0.0782 kip/ft
Wind uplift Case A: 0.0782 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1087 kip/ft



Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	11.51645269	0.334	PASS
Material Yield	34.5	11.51645269	0.334	PASS
Material Strength	37	11.51645269	0.311	PASS

Member 1, ULS: 1. 1.4D



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	0.0080	1.8712	0.0380	0.0861	-0.0123	-0.0321
ULS: 2. D + L	0.0080	1.8712	0.0380	0.0861	-0.0123	-0.0321
ULS: 3. D + (S or Lr or R)	0.0147	3.1422	0.0702	0.1589	-0.0227	-0.0797
ULS: 3. D + (S or Lr or R)	0.0080	1.8712	0.0380	0.0861	-0.0123	-0.0321
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	2.8245	0.0622	0.1407	-0.0201	-0.0678
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0080	1.8712	0.0380	0.0861	-0.0123	-0.0321
ULS: 5b. D + 0.7E	0.0080	1.8712	0.0380	0.0861	-0.0123	-0.0321
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0131	2.8245	0.0622	0.1407	-0.0201	-0.0678
ULS: 8. 0.6D + 0.7E	0.0048	1.1227	0.0228	0.0516	-0.0074	-0.0192
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1623	5.6095	0.1625	0.3522	-0.1921	17.4838
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1623	5.6095	0.1625	0.3522	-0.1921	17.4838
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8671	-1.3324	-0.0659	-0.1357	0.1381	-14.4096
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.5671	-0.8033	-0.0635	-0.1301	0.1384	-19.7557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6147	5.6282	0.1555	0.3403	-0.1550	13.0691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6147	5.6282	0.1555	0.3403	-0.1550	13.0691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4074	0.4218	-0.0158	-0.0257	0.0926	-10.8510
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1824	0.8186	-0.0139	-0.0214	0.0928	-14.8605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6198	4.6750	0.1314	0.2857	-0.1471	13.1048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6198	4.6750	0.1314	0.2857	-0.1471	13.1048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4023	-0.5315	-0.0399	-0.0803	0.1005	-10.8152
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1773	-0.1347	-0.0381	-0.0760	0.1007	-14.8248
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1655	4.8610	0.1473	0.3178	-0.1872	17.4966
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1655	4.8610	0.1473	0.3178	-0.1872	17.4966
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8639	-2.0809	-0.0811	-0.1701	0.1430	-14.3968
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5639	-1.5518	-0.0787	-0.1645	0.1433	-19.7429

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	9.1117
Shear X	-3.6172
Shear Z	0.2705
Moment X	0.5867
Moment Y (Twist)	0.3214
Moment Z	33.3645

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	5.6282
Shear X	-2.1655
Shear Z	0.1625
Moment X	0.3522
Moment Y (Twist)	0.1921
Moment Z	19.7557

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	-0.0080	2.1136	-0.0043	-0.0096	0.0055	0.0855
ULS: 2. D + L	-0.0080	2.1136	-0.0043	-0.0096	0.0055	0.0855
ULS: 3. D + (S or Lr or R)	-0.0147	3.5886	-0.0079	-0.0177	0.0102	0.1374
ULS: 3. D + (S or Lr or R)	-0.0080	2.1136	-0.0043	-0.0096	0.0055	0.0855
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0130	3.2198	-0.0070	-0.0156	0.0090	0.1244

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0080	2.1136	-0.0043	-0.0096	0.0055	0.0855
ULS: 5b. D + 0.7E	-0.0080	2.1136	-0.0043	-0.0096	0.0055	0.0855
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0130	3.2198	-0.0070	-0.0156	0.0090	0.1244
ULS: 8. 0.6D + 0.7E	-0.0048	1.2681	-0.0026	-0.0057	0.0033	0.0513
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5101	6.4682	-0.0064	-0.0162	-0.0000	20.1455
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5101	6.4682	-0.0064	-0.0162	-0.0000	20.1455
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1379	-1.6197	-0.0011	-0.0013	0.0079	-16.3071
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7704	-0.9926	-0.0103	-0.0209	0.0238	-22.1791
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8897	6.4859	-0.0086	-0.0206	0.0049	15.1695
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8897	6.4859	-0.0086	-0.0206	0.0049	15.1695
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5964	0.4199	-0.0046	-0.0094	0.0108	-12.1700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3208	0.8902	-0.0115	-0.0241	0.0227	-16.5740
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8846	5.3796	-0.0059	-0.0145	0.0014	15.1305
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8846	5.3796	-0.0059	-0.0145	0.0014	15.1305
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6014	-0.6864	-0.0019	-0.0034	0.0073	-12.2090
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3258	-0.2161	-0.0088	-0.0181	0.0192	-16.6130
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5070	5.6228	-0.0047	-0.0123	-0.0022	20.1114
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5070	5.6228	-0.0047	-0.0123	-0.0022	20.1114
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1411	-2.4651	0.0006	0.0025	0.0057	-16.3413
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7736	-1.8380	-0.0086	-0.0171	0.0216	-22.2133

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	10.5314
Shear X	-4.1826
Shear Z	-0.0178
Moment X	-0.0358
Moment Y (Twist)	0.0409
Moment Z	37.5079

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.4859
Shear X	-2.5101
Shear Z	-0.0115
Moment X	-0.0241
Moment Y (Twist)	0.0238
Moment Z	22.2133

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	-0.0080	2.1136	0.0043	0.0096	-0.0055	0.0854
ULS: 2. D + L	-0.0080	2.1136	0.0043	0.0096	-0.0055	0.0854
ULS: 3. D + (S or Lr or R)	-0.0147	3.5886	0.0079	0.0176	-0.0101	0.1374
ULS: 3. D + (S or Lr or R)	-0.0080	2.1136	0.0043	0.0096	-0.0055	0.0854
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0130	3.2198	0.0070	0.0156	-0.0090	0.1244
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0080	2.1136	0.0043	0.0096	-0.0055	0.0854
ULS: 5b. D + 0.7E	-0.0080	2.1136	0.0043	0.0096	-0.0055	0.0854
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0130	3.2198	0.0070	0.0156	-0.0090	0.1244
ULS: 8. 0.6D + 0.7E	-0.0048	1.2681	0.0026	0.0057	-0.0033	0.0513
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5101	6.4682	0.0064	0.0162	0.0000	20.1455
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5101	6.4682	0.0064	0.0162	0.0000	20.1455
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1379	-1.6197	0.0011	0.0013	-0.0079	-16.3071
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7704	-0.9926	0.0103	0.0209	-0.0238	-22.1791

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	-1.8897	6.4859	0.0086	0.0206	-0.0048	15.1695
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8897	6.4859	0.0086	0.0206	-0.0048	15.1695
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5964	0.4199	0.0046	0.0094	-0.0108	-12.1700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3208	0.8902	0.0115	0.0241	-0.0227	-16.5740
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8846	5.3796	0.0059	0.0145	-0.0014	15.1305
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8846	5.3796	0.0059	0.0145	-0.0014	15.1305
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6014	-0.6864	0.0019	0.0034	-0.0073	-12.2090
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3258	-0.2161	0.0088	0.0181	-0.0192	-16.6130
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5070	5.6228	0.0047	0.0123	0.0022	20.1114
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5070	5.6228	0.0047	0.0123	0.0022	20.1114
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1411	-2.4651	-0.0006	-0.0025	-0.0057	-16.3413
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7736	-1.8380	0.0086	0.0171	-0.0216	-22.2133

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	10.5314
Shear X	-4.1826
Shear Z	0.0178
Moment X	0.0361
Moment Y (Twist)	0.0408
Moment Z	37.5079

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.4859
Shear X	-2.5101
Shear Z	0.0115
Moment X	0.0241
Moment Y (Twist)	0.0238
Moment Z	22.2133

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	0.0080	1.8712	-0.0381	-0.0861	0.0123	-0.0321
ULS: 2. D + L	0.0080	1.8712	-0.0381	-0.0861	0.0123	-0.0321
ULS: 3. D + (S or Lr or R)	0.0147	3.1422	-0.0702	-0.1589	0.0228	-0.0797
ULS: 3. D + (S or Lr or R)	0.0080	1.8712	-0.0381	-0.0861	0.0123	-0.0321
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0130	2.8245	-0.0622	-0.1407	0.0201	-0.0678
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0080	1.8712	-0.0381	-0.0861	0.0123	-0.0321
ULS: 5b. D + 0.7E	0.0080	1.8712	-0.0381	-0.0861	0.0123	-0.0321
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0130	2.8245	-0.0622	-0.1407	0.0201	-0.0678
ULS: 8. 0.6D + 0.7E	0.0048	1.1227	-0.0228	-0.0516	0.0074	-0.0192
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1623	5.6095	-0.1625	-0.3522	0.1921	17.4838
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1623	5.6095	-0.1625	-0.3522	0.1921	17.4838
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8671	-1.3324	0.0659	0.1357	-0.1381	-14.4096
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.5671	-0.8033	0.0635	0.1300	-0.1383	-19.7557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6147	5.6282	-0.1555	-0.3403	0.1550	13.0691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6147	5.6282	-0.1555	-0.3403	0.1550	13.0691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4074	0.4218	0.0158	0.0256	-0.0926	-10.8510
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1824	0.8186	0.0139	0.0214	-0.0928	-14.8605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6198	4.6750	-0.1314	-0.2857	0.1472	13.1048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6198	4.6750	-0.1314	-0.2857	0.1472	13.1048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4023	-0.5315	0.0399	0.0803	-0.1005	-10.8152
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1773	-0.1347	0.0381	0.0760	-0.1007	-14.8248

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1655	4.8610	-0.1473	-0.3178	0.1872	17.4966
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1655	4.8610	-0.1473	-0.3178	0.1872	17.4966
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8639	-2.0809	0.0811	0.1701	-0.1430	-14.3968
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5639	-1.5518	0.0787	0.1645	-0.1432	-19.7429

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.1117
Shear X	-3.6172
Shear Z	-0.2705
Moment X	-0.5868
Moment Y (Twist)	0.3216
Moment Z	33.3653

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	5.6282
Shear X	-2.1655
Shear Z	-0.1625
Moment X	-0.3522
Moment Y (Twist)	0.1921
Moment Z	19.7557

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : MTSOLAR_7AJLJ4L53AC4B
 Designer : MT_SKYCIV AutoDesigner
 Company : MT Solar
 Units : Imperial
 Notes : Snow loads based on monoslope structure



Project Data

The structure is located in **4675 KY-181, Greenville, KY 42345, USA** categorized as **Exposure C** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Moreover, the structure is classified as **Risk Category I**. The location is elevated at **487.86 ft** above mean sea level.

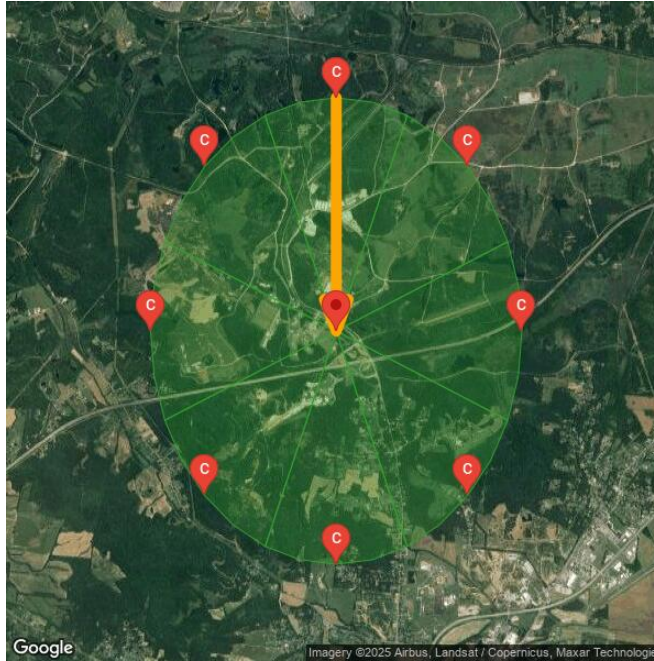


Figure 1. Site location.

Parameter	Value
Building Length, L	9.85 ft
Building Width, B	86.25 ft
Mean Roof Height, h	7.84 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.00°
Structure Type	Main Wind Force Resisting System (MWFRS)
Wind Blockage	Empty Under

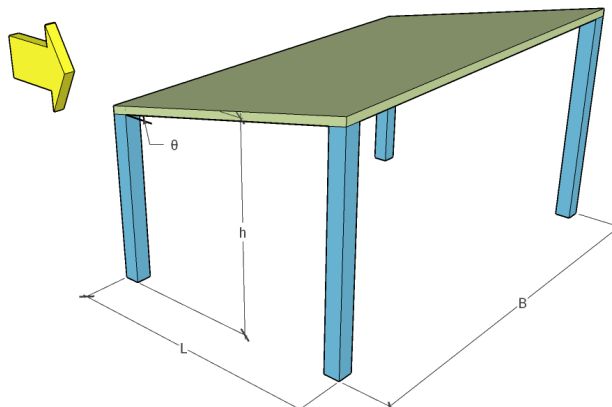


Figure 2. Building parameters.

Figure 26.5-1

Basic Wind Speed, V

Wind speed for the address is **99 mph** for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.

$V = 99$ mph

Table 26.6-1	<p>Wind Directionality Factor, K_d</p> <p>$K_d = 0.85$ - Wind Directionality Factor For buildings</p>	$K_d = 0.85$																																																								
Section 26.8.1	<p>Topographic Factor, K_{zt}</p> <p>$K_{zt} = 1$ - Topographic Factor For the selected wind source direction, either the terrain is relatively a flat surface or the structure is outside the local topographic zones.</p> <p>For calculating the topographic factor, the detected topography for the selected wind source direction is Flat.</p>	$K_{zt} = 1$																																																								
Section 26.9	<p>Ground Elevation Factor, K_e</p> <p>K_e - Ground Elevation Factor</p> $K_e = e^{-0.000362 E}$ $K_e = 0.98249$ <p>Where E = Site Elevation = 487.86 ft</p>																																																									
Section 26.9	$K_e = 0.98249$ - Ground Elevation Factor	$K_e = 0.98249$																																																								
Section 26.10	<p>Velocity Pressure Exposure Coefficient, K_z</p> <p>K_z - Velocity Pressure Exposure Coefficient For $z < 15ft$</p> $K_z = 2.01 \times (15/z_g)^{2/\alpha}$																																																									
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Table 26.11-1	$\alpha = 9.5$	$\alpha = 9.5$																																																								
Table 26.11-1	$z_g = 900$ ft	$z_g = 900$ ft																																																								
Section 26.10	<p>K_z - Velocity Pressure Exposure Coefficient For $z < 15ft$</p> $K_z = 2.01 (15/z_g)^{2/\alpha}$ $K_z = 0.84888$ <p>Where $z_g = 900$ ft $\alpha = 9.5$</p> <table border="1"> <thead> <tr> <th>Level</th> <th>Elevation (ft)</th> <th>K_z</th> </tr> </thead> <tbody> <tr> <td>h</td> <td>7.84</td> <td>0.849</td> </tr> </tbody> </table>	Level	Elevation (ft)	K_z	h	7.84	0.849																																																			
Level	Elevation (ft)	K_z																																																								
h	7.84	0.849																																																								
Section 26.10.2	<p>Velocity Pressure, q_h</p> <p>For the selected wind source direction.</p> <p>q_h - Velocity Pressure at h</p> $q_h = 0.00256 K_z K_{zt} K_d K_e V^2$ $q_h = 17.787$ psf <p>Where K_z = Velocity Pressure Exposure Coefficient = 0.84888 K_{zt} = Topographic Factor = 1 K_d = Wind Directionality Factor = 0.85 V = Basic Wind Speed = 99 mi/h K_e = Ground Elevation Factor = 0.98249</p>																																																									
Section 26.8	<p>Velocity Pressure for All Directions</p> <p>K_{zt} - Topographic Factor</p> $K_{zt} = (1 + K_1 \times K_2 \times K_3)^2 \geq 1.0$																																																									
Section 26.10	<p>K_z - Velocity Pressure Exposure Coefficient For $15ft \leq z \leq z_g$</p> $K_z = 2.01 \times (z/z_g)^{2/\alpha}$																																																									
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Section 26.10.2	<p>q_h - Velocity Pressure at h</p> $q_h = 0.00256 \times K_{z,h} \times K_{zt} \times K_d \times K_e \times V^2$ <table border="1"> <thead> <tr> <th>Direction</th> <th>Exposure Category</th> <th>K_z @ $h = 7.844ft$</th> <th>K_{zt}</th> <th>K_d</th> <th>K_e</th> <th>V (mph)</th> <th>q_h (psf)</th> </tr> </thead> <tbody> <tr> <td>N</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> <tr> <td>NE</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> <tr> <td>E</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> <tr> <td>SE</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> <tr> <td>S</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> <tr> <td>SW</td> <td>C</td> <td>0.849</td> <td>1.000</td> <td>0.850</td> <td>0.982</td> <td>99.000</td> <td>17.787</td> </tr> </tbody> </table>	Direction	Exposure Category	K_z @ $h = 7.844ft$	K_{zt}	K_d	K_e	V (mph)	q_h (psf)	N	C	0.849	1.000	0.850	0.982	99.000	17.787	NE	C	0.849	1.000	0.850	0.982	99.000	17.787	E	C	0.849	1.000	0.850	0.982	99.000	17.787	SE	C	0.849	1.000	0.850	0.982	99.000	17.787	S	C	0.849	1.000	0.850	0.982	99.000	17.787	SW	C	0.849	1.000	0.850	0.982	99.000	17.787	
Direction	Exposure Category	K_z @ $h = 7.844ft$	K_{zt}	K_d	K_e	V (mph)	q_h (psf)																																																			
N	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			
NE	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			
E	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			
SE	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			
S	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			
SW	C	0.849	1.000	0.850	0.982	99.000	17.787																																																			

W	C	0.849	1.000	0.850	0.982	99.000	17.787
NW	C	0.849	1.000	0.850	0.982	99.000	17.787

Net Pressure Coefficients, C_N

Figure 27.3-4 to 27.3-7

The net pressure coefficients, C_N , are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.

Direction	Surface	C_N Case A	C_N Case B
0	Windward	-1.800	-2.500
	Leeward		-0.500
180	Windward	2.100	2.600
	Leeward		1.000
90	≤ h from windward edge	-0.800	0.800
	h to 2h from windward edge	-0.600	0.500
	> 2h from windward edge	-0.300	0.300

Gust Effect Factor, G

Section 26.11.1

$G = 0.85$ - Gust Effect Factor
The structure is assumed to be rigid.

$G = 0.85$

Design Wind Pressures (MWFRS)

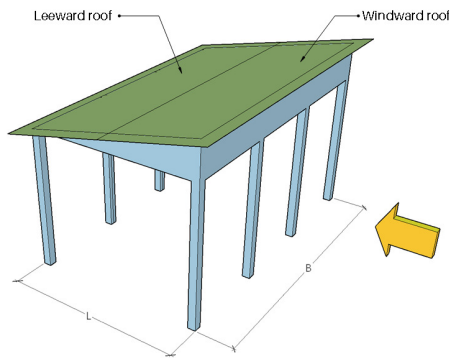
Section 27.3.2

p - Design Wind Pressure
For open buildings

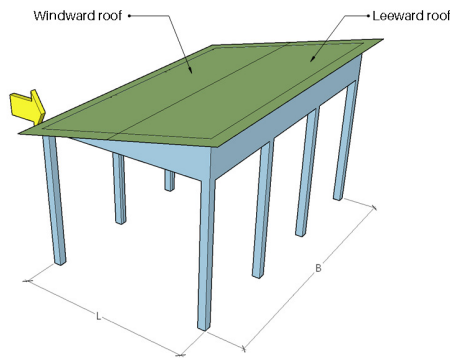
$$p = q_h \times G \times C_N$$

For Wind Pressure - 0°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
0	Windward	17.787	0.850	-1.800	-2.500	-27.214	-37.798
	Leeward	17.787	0.850		-0.500	-27.214	-7.560
180	Windward	17.787	0.850	2.100	2.600	31.750	39.310
	Leeward	17.787	0.850		1.000	31.750	15.119



Wind along L - 0°



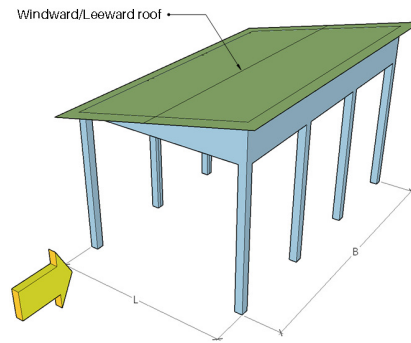
Wind along L - 180°

Service Wind Pressure - 0°/180°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	-16.329	-22.679
	Leeward	-16.329	-4.536
180	Windward	19.050	23.586
	Leeward	19.050	9.071

For Wind Pressure - 90°

Direction	Surface	q_h (psf)	G	C_{N} Case A	C_{N} Case B	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	17.787	0.850	-0.800	0.800	-12.095	12.095
	h to 2h from windward edge	17.787	0.850	-0.600	0.500	-9.071	7.560
	> 2h from windward edge	17.787	0.850	-0.300	0.300	-4.536	4.536



Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-7.257	7.257
	h to 2h from windward edge	-5.443	4.536
	> 2h from windward edge	-2.721	2.721

Section 27.3.2
Section 28.3.5

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted and be reduced based on the actual solidity ratio ϕ and number of frames n - See Figure 28.3-2.

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$

Section 27.3.2
Section 28.3.5

K_B - Frame Width Factor

For $L < 100ft$, $K_B = 1.8 - 0.01L$. Otherwise, $K_B = 0.8$.

$$K_B = 1.8 - 0.01 * L \leq 0.8$$

$$K_B = 1.7015$$

Where L = Building Length = 9.851 ft

Section 28.3.5

K_S - Shielding Factor

$$K_S = 0.6 + 0.073 \times (n - 1) + (1.25 \times \phi^{1.8})$$

Section 28.3.5

$K_S = 1$ - Shielding Factor

Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n .

$K_S = 1$

Snow Load Calculations based on ASCE 7-16

Design Information :

Project Name : MTSOLAR_7AJLJ4L53AC4B
 Designer : MT_SKYCIV AutoDesigner
 Company : MT Solar
 Units : Imperial
 Notes : Snow loads based on monoslope structure



Project Data

The structure is located in **4675 KY-181, Greenville, KY 42345, USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **488 ft** above mean sea level.

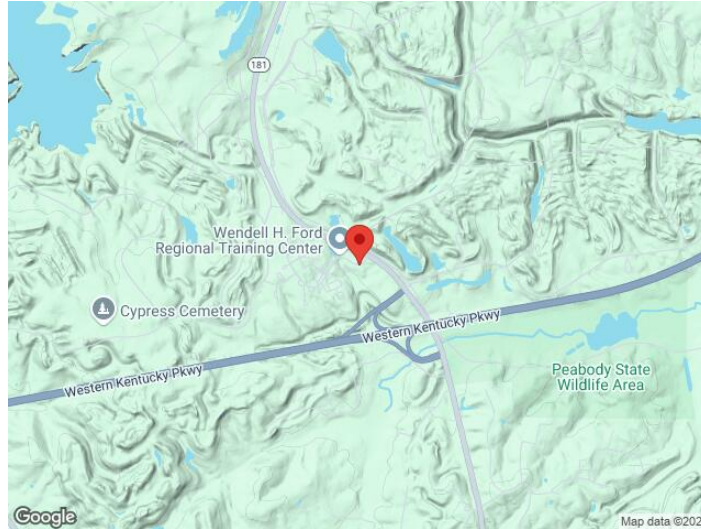


Figure 1. Site location.

Parameter	Value
Building Length, <i>L</i>	9.851 ft
Building Width, <i>B</i>	86.250 ft
Mean Roof Height, <i>h</i>	7.844 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.000°

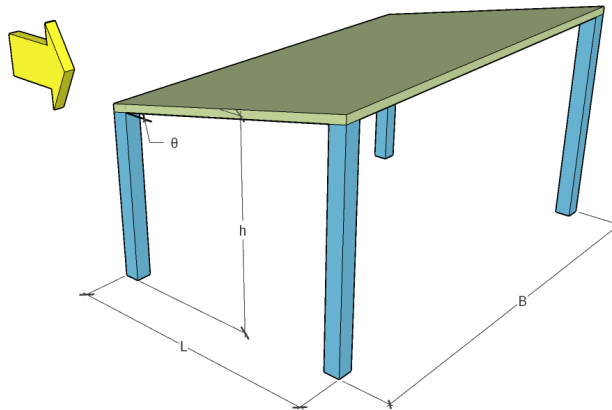


Figure 2. Building parameters.

Figure 7.6-2

S

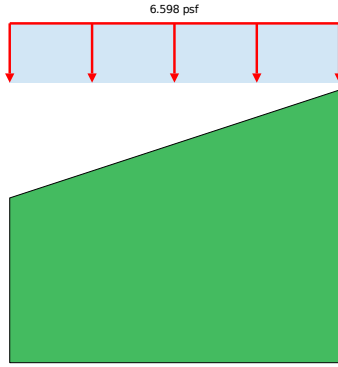
Where roof slope is equal to $\frac{1}{S}$

$$S = \frac{1}{\tan\theta}$$

$$S = 1.7321$$

Where θ = Angle of slope of roof = 30 °

Ground Snow Load, *D_s*



Section 7.3.4

Minimum Roof Snow Load, p_m

$p_m = 0$ psf - Minimum Roof Snow Load
For monoslope, gable, or hip roof with pitch angle $\theta \geq 15^\circ$.

$p_m = 0$ psf

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: MTSOLAR_7AJLJ4L53AC4B
 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				
7	6in Pipe Sch 40	6.63	0.28				

Section Dimensions							

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	

Section Dimensions							

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_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

314	19	4.88	4.00	0	9,1.32,1.09,1.09,1.09,1.07	0	0	1
315	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.14,1.14,1.14,1.15,1.14,1.14,1.14,1.15,1.14,1.14,1.17,1.20,1.14,1.14,1.15,1.16,1.14,1.14,1.14,1.15	300	200	1
316	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.77,1.13,1.13,1.13,1.42,1.13,1.13,1.19,1.15,1.13,1.13,1.13,1.20,1.13,1.13,1.13,1.12	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	251.16	142.53	42.30	42.30	75.35	75.35
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	24.61	6.12	40.24	43.62
14	133.20	85.85	24.54	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62
101	251.16	142.53	42.30	42.30	75.35	75.35
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	23.61	6.12	40.24	43.62
114	133.20	85.85	23.38	6.12	40.24	43.62
115	133.20	46.28	12.18	6.12	40.24	43.62
116	133.20	46.28	12.35	6.12	40.24	43.62
201	251.16	142.53	42.30	42.30	75.35	75.35
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	23.61	6.12	40.24	43.62
214	133.20	85.85	23.33	6.12	40.24	43.62
215	133.20	46.28	12.21	6.12	40.24	43.62
216	133.20	46.28	12.11	6.12	40.24	43.62
301	251.16	142.53	42.30	42.30	75.35	75.35
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	32.95	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	32.95	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	24.61	6.12	40.24	43.62
314	133.20	85.85	24.51	6.12	40.24	43.62
315	133.20	46.28	12.29	6.12	40.24	43.62
316	133.20	46.28	12.14	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	0.064	0.789	0.036	0.048	0.004	0.797	#16	0.440	Not Required	Pass
2	0.002	0.319	0.155	0.070	0.029	0.475	#13	0.035	Not Required	Pass
3	0.004	0.526	0.021	0.052	0.003	0.531	#13	0.045	Not Required	Pass
4	0.004	0.527	0.079	0.053	0.017	0.574	#13	0.080	Not Required	Pass
5	0.004	0.326	0.073	0.052	0.019	0.334	#13	0.074	Not Required	Pass
6	0.005	0.600	0.037	0.061	0.005	0.629	#13	0.045	Not Required	Pass
7	0.005	0.372	0.102	0.060	0.026	0.392	#13	0.074	Not Required	Pass
8	0.001	0.061	0.109	0.043	0.009	0.122	#24	0.095	Not Required	Pass
9	0.009	0.061	0.047	0.002	0.001	0.110	#13	0.204	Not Required	Pass
10	0.005	0.584	0.099	0.059	0.021	0.604	#13	0.080	Not Required	Pass
11	0.002	0.055	0.110	0.044	0.009	0.129	#21	0.095	Not Required	Pass
12	0.002	0.389	0.170	0.080	0.032	0.559	#13	0.053	Not Required	Pass
13	0.003	0.224	0.230	0.054	0.011	0.358	#21	0.286	Not Required	Pass
14	0.005	0.220	0.228	0.053	0.011	0.344	#21	0.190	Not Required	Pass
15	0.000	0.082	0.091	0.028	0.005	0.152	#21	Not Required	Not Required	Pass
16	0.000	0.082	0.091	0.028	0.005	0.152	#21	Not Required	Not Required	Pass
101	0.074	0.887	0.002	0.056	0.000	0.891	#16	0.440	Not Required	Pass
102	0.002	0.412	0.189	0.087	0.035	0.601	#13	0.035	Not Required	Pass
103	0.005	0.647	0.026	0.065	0.001	0.667	#13	0.045	Not Required	Pass
104	0.005	0.656	0.097	0.066	0.020	0.698	#13	0.080	Not Required	Pass
105	0.005	0.401	0.102	0.064	0.026	0.419	#13	0.074	Not Required	Pass
106	0.005	0.650	0.026	0.065	0.002	0.666	#13	0.045	Not Required	Pass
107	0.005	0.403	0.098	0.065	0.025	0.421	#13	0.074	Not Required	Pass
108	0.002	0.068	0.103	0.044	0.008	0.153	#21	0.095	Not Required	Pass
109	0.010	0.063	0.040	0.001	0.000	0.104	#13	0.204	Not Required	Pass
110	0.005	0.648	0.095	0.065	0.020	0.698	#13	0.080	Not Required	Pass

110	0.003	0.046	0.093	0.003	0.020	0.008	#13	0.000	Not Required	Pass
111	0.001	0.060	0.104	0.044	0.008	0.151	#21	0.095	Not Required	Pass
112	0.002	0.409	0.189	0.087	0.035	0.599	#13	0.035	Not Required	Pass
113	0.003	0.246	0.230	0.056	0.011	0.401	#21	0.286	Not Required	Pass
114	0.006	0.265	0.229	0.057	0.011	0.410	#21	0.286	Not Required	Pass
115	0.004	0.411	0.121	0.045	0.009	0.490	#13	0.601	Not Required	Pass
116	0.003	0.404	0.123	0.046	0.009	0.484	#13	0.601	Not Required	Pass
201	0.074	0.887	0.002	0.056	0.000	0.891	#16	0.440	Not Required	Pass
202	0.002	0.409	0.189	0.087	0.035	0.599	#13	0.035	Not Required	Pass
203	0.005	0.650	0.026	0.065	0.002	0.666	#13	0.045	Not Required	Pass
204	0.005	0.648	0.095	0.065	0.020	0.688	#13	0.080	Not Required	Pass
205	0.005	0.403	0.098	0.065	0.025	0.421	#13	0.074	Not Required	Pass
206	0.005	0.647	0.026	0.065	0.001	0.667	#13	0.045	Not Required	Pass
207	0.005	0.401	0.102	0.064	0.026	0.419	#13	0.074	Not Required	Pass
208	0.001	0.065	0.110	0.046	0.009	0.157	#21	0.095	Not Required	Pass
209	0.010	0.063	0.040	0.001	0.000	0.104	#13	0.204	Not Required	Pass
210	0.005	0.656	0.097	0.066	0.020	0.698	#13	0.080	Not Required	Pass
211	0.002	0.053	0.111	0.045	0.009	0.153	#21	0.095	Not Required	Pass
212	0.002	0.412	0.189	0.087	0.035	0.601	#13	0.035	Not Required	Pass
213	0.003	0.246	0.230	0.056	0.011	0.401	#21	0.286	Not Required	Pass
214	0.006	0.265	0.229	0.057	0.011	0.410	#21	0.286	Not Required	Pass
215	0.004	0.371	0.121	0.044	0.008	0.449	#13	0.601	Not Required	Pass
216	0.004	0.347	0.122	0.044	0.008	0.427	#13	0.601	Not Required	Pass
301	0.064	0.789	0.036	0.048	0.004	0.797	#16	0.440	Not Required	Pass
302	0.002	0.389	0.170	0.080	0.032	0.559	#13	0.053	Not Required	Pass
303	0.005	0.600	0.037	0.061	0.005	0.629	#13	0.045	Not Required	Pass
304	0.005	0.584	0.099	0.059	0.021	0.604	#13	0.080	Not Required	Pass
305	0.005	0.372	0.102	0.060	0.026	0.392	#13	0.074	Not Required	Pass
306	0.004	0.526	0.021	0.052	0.003	0.531	#13	0.045	Not Required	Pass
307	0.004	0.326	0.073	0.052	0.019	0.334	#13	0.074	Not Required	Pass
308	0.000	0.082	0.091	0.028	0.005	0.152	#21	Not Required	Not Required	Pass
309	0.009	0.061	0.047	0.002	0.001	0.110	#13	0.204	Not Required	Pass
310	0.004	0.527	0.079	0.053	0.017	0.574	#13	0.080	Not Required	Pass
311	0.000	0.082	0.091	0.028	0.005	0.152	#21	Not Required	Not Required	Pass
312	0.002	0.319	0.155	0.070	0.029	0.475	#13	0.035	Not Required	Pass
313	0.003	0.224	0.230	0.054	0.011	0.358	#21	0.190	Not Required	Pass
314	0.005	0.220	0.228	0.053	0.011	0.344	#21	0.286	Not Required	Pass
315	0.004	0.415	0.121	0.044	0.009	0.492	#13	0.601	Not Required	Pass
316	0.003	0.414	0.122	0.043	0.009	0.494	#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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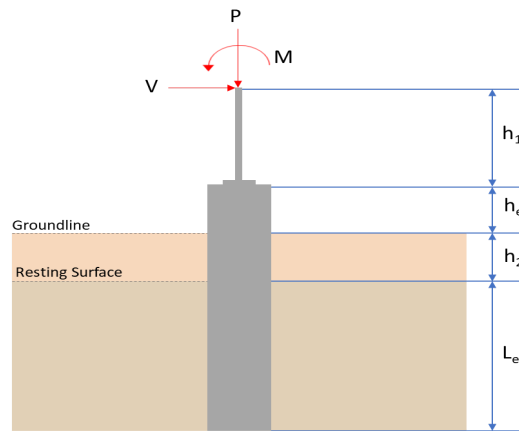
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sedimentary & foliated rock	4000.000	400.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.628	9.112
V_x (kip)	-2.166	-3.617
V_z (kip)	0.162	0.271
M_x (kipft)	0.352	0.587
M_z (kipft)	19.756	33.365

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,

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

$$H_o = \frac{(-2.166 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.722 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.756 \text{ kipft}) + ((-2.166 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.5853 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5307 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.162 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.054 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.352 \text{ kipft}) + ((0.162 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.11733 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1241 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5307 \text{ ft}), (2.1241 \text{ ft})]$$

$$L_{e,req} = 5.531 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.531 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92183$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.628 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.7962 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.7962 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.19905$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.722 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.5853 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5853 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.5853 \text{ kipft/ft})) + (4 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1524 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.5853 \text{ kipft/ft})) + (3 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (6.5853 \text{ kipft/ft})) + (2 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.5254 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.5853 \text{ kipft/ft})) + ((-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 2.314 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.1524 \text{ ft})}{2}$$

$$p_a = 0.83049 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.5254 \text{ kip/ft}^2)}{(0.83049 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63264$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 2.4 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(2.314 \text{ kip/ft}^2)}{(2.4 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96417$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.054 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11733 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11733 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.054 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.11733 \text{ kipft/ft})) + (4 \times (0.054 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.324 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.11733 \text{ kipft/ft})) + (3 \times (0.054 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.11733 \text{ kipft/ft})) + (2 \times (0.054 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.067979 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.11733 \text{ kipft/ft})) + ((0.054 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.14626 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.324 \text{ ft})}{2}$$

$$p_a = 0.8648 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.067979 \text{ kip/ft}^2)}{(0.8648 \text{ kip/ft}^2)}$$

(0.0000 kip/ft)

$$Ratio = 0.078606$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 2.4 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

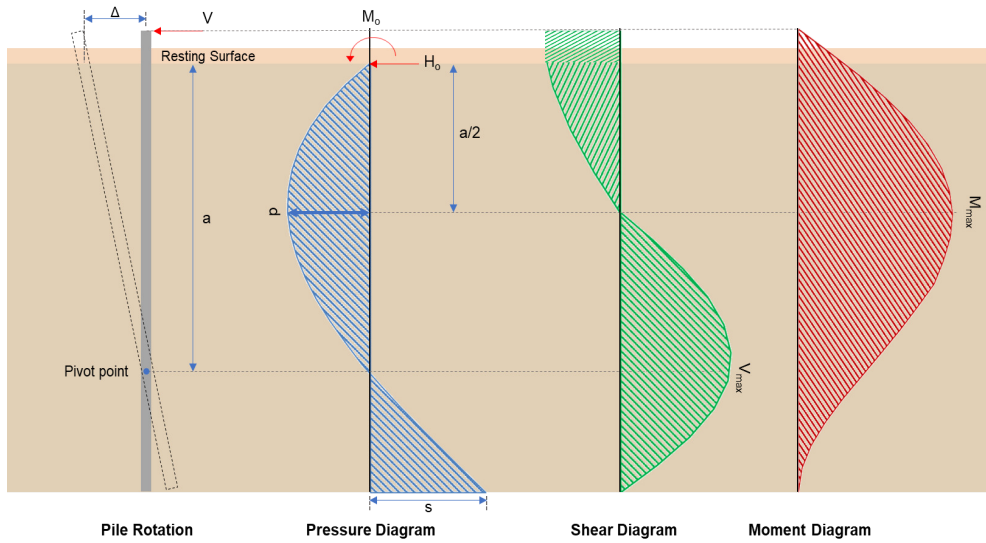
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.14626 \text{ kip/ft}^2)}{(2.4 \text{ kip/ft}^2)}$$

$$Ratio = 0.060943$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.617 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.2057 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.365 \text{ kipft}) + ((-3.617 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 11.122 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.122 \text{ kipft/ft})}{(-1.2057 \text{ kip/ft})}$$

$$E = 9.2245 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.122 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-1.2057 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (11.122 \text{ kipft/ft})) + (4 \times (-1.2057 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1512 \text{ ft}$$

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

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

$$V_{max} = ((-1.2057 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1512 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1512 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.225 \text{ kip}$$

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

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

$$M_{max} = ((-1.2057 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(9.2245 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.707 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.271 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.090333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.587 \text{ kipft}) + ((0.271 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19567 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19567 \text{ kipft/ft})}{(0.090333 \text{ kip/ft})}$$

$$E = 2.1661 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19567 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.090333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.19567 \text{ kipft/ft})) + (4 \times (0.090333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3244 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((0.090333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3244 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3244 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.35459 \text{ kip}$$

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

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

$$M_{max} = ((0.090333 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.1661 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91933 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.112 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.088 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.088 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{1.8322 \text{ in}^2}{1.8408 \text{ in}^2}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.112 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0072669$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.112 \text{ kip} \rightarrow 9112 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9112 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.985 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.985 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.985 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.985 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.201 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.225 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.225 \text{ kip})}{(74.201 \text{ kip})}$$

$$Ratio = 0.16475$$

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

Considering z-direction:

$V_{max} = 0.35459 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.35459 \text{ kip})}{(74.201 \text{ kip})}$$

$$Ratio = 0.0047787$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.707 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.707 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.55955$$

Status: **PASS**
Ratio: **0.560**

Considering z-direction:

$M_{max} = 0.91933 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.91933 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.014822$$

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

REFERENCES	CALCULATIONS	RESULTS
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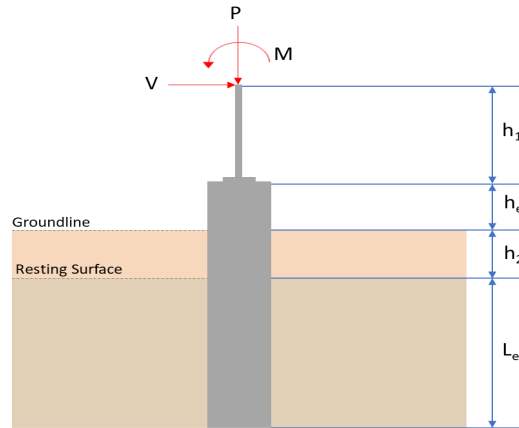
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sedimentary & foliated rock	4000.000	400.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.628	9.112
V_x (kip)	-2.166	-3.617
V_z (kip)	-0.162	-0.271
M_x (kipft)	-0.352	-0.587
M_z (kipft)	19.756	33.365

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,

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

$$H_o = \frac{(-2.166 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.722 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.756 \text{ kipft}) + ((-2.166 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.5853 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5307 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.162 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.054 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.352 \text{ kipft}) + ((-0.162 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.11733 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4144 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5307 \text{ ft}), (1.4144 \text{ ft})]$$

$$L_{e,req} = 5.531 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.531 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92183$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.628 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.7962 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.7962 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.19905$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.722 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.5853 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5853 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.5853 \text{ kipft/ft})) + (4 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1524 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.5853 \text{ kipft/ft})) + (3 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (6.5853 \text{ kipft/ft})) + (2 \times (-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.5254 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.5853 \text{ kipft/ft})) + ((-0.722 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 2.314 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.1524 \text{ ft})}{2}$$

$$p_a = 0.83049 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.5254 \text{ kip/ft}^2)}{(0.83049 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63264$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 2.4 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(2.314 \text{ kip/ft}^2)}{(2.4 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96417$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.054 \text{ kip/ft}$ - Lateral force per length of pile.

$M_o = 0.11733 \text{ kipft/ft}$ - Overturning moment per length of pile.

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11733 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.054 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.11733 \text{ kipft/ft})) + (4 \times (-0.054 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.324 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.11733 \text{ kipft/ft})) + (3 \times (-0.054 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.11733 \text{ kipft/ft})) + (2 \times (-0.054 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.027933 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.11733 \text{ kipft/ft})) + ((-0.054 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = -0.023388 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.324 \text{ ft})}{2}$$

$$p_a = 0.8648 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.027933 \text{ kip/ft}^2)}{(0.8648 \text{ kip/ft}^2)}$$

$$\left(\frac{0.0023 \text{ kip/ft}^2}{2.4} \right)$$

$$\text{Ratio} = -0.032299$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 2.4 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

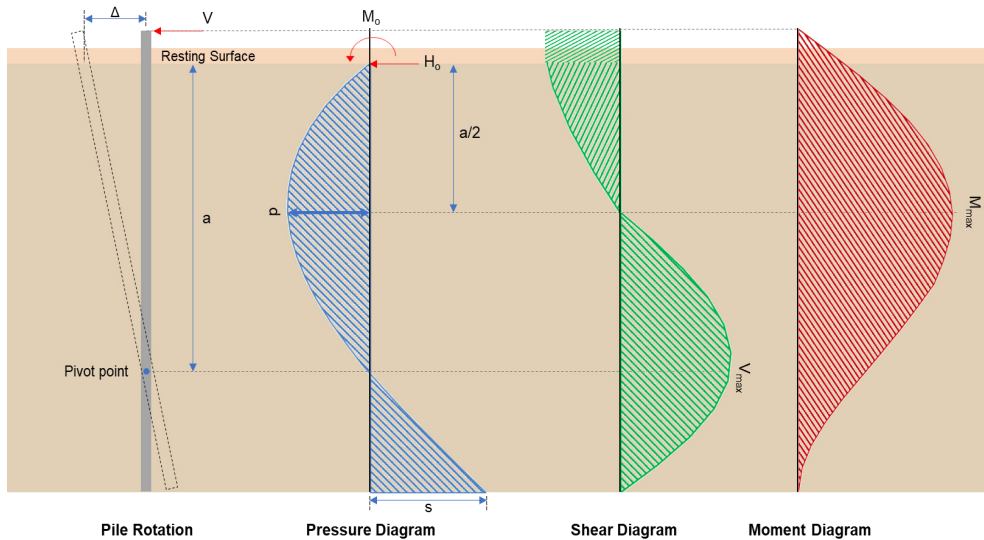
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.023388 \text{ kip/ft}^2)}{(2.4 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.009745$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.617 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.2057 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.365 \text{ kipft}) + ((-3.617 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 11.122 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.122 \text{ kipft/ft})}{(-1.2057 \text{ kip/ft})}$$

$$E = 9.2245 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.122 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-1.2057 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (11.122 \text{ kipft/ft})) + (4 \times (-1.2057 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1512 \text{ ft}$$

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

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

$$V_{max} = ((-1.2057 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1512 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1512 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.225 \text{ kip}$$

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

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

$$M_{max} = ((-1.2057 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(9.2245 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.2245 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1512 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.707 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.271 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.090333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.587 \text{ kipft}) + ((-0.271 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19567 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19567 \text{ kipft/ft})}{(-0.090333 \text{ kip/ft})}$$

$$E = 2.1661 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19567 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.090333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.19567 \text{ kipft/ft})) + (4 \times (-0.090333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3244 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.090333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3244 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3244 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.35459 \text{ kip}$$

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

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

$$M_{max} = ((-0.090333 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.1661 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1661 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3244 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91933 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.112 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.088 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.088 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{1.8322 \text{ in}^2}{1.8408 \text{ in}^2}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.112 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0072669$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.112 \text{ kip} \rightarrow 9112 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9112 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.985 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.985 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.985 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.985 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.201 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.225 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(12.225 \text{ kip})}{(74.201 \text{ kip})}$$

$$Ratio = 0.16475$$

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

Considering z-direction:

$V_{max} = 0.35459 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.35459 \text{ kip})}{(74.201 \text{ kip})}$$

$$Ratio = 0.0047787$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.707 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.707 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.55955$$

Status: **PASS**
Ratio: **0.560**

Considering z-direction:

$M_{max} = 0.91933 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.91933 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.014822$$

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

REFERENCES	CALCULATIONS	RESULTS
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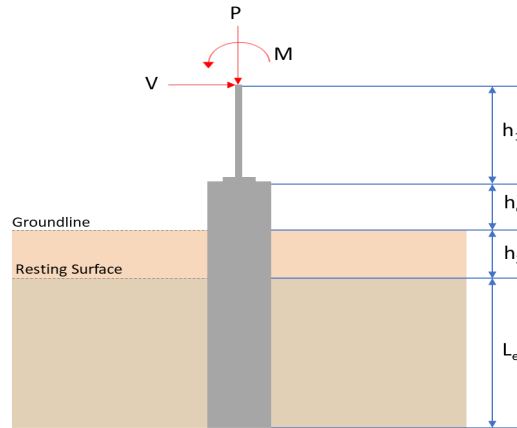
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6.25$ 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	Sedimentary & foliated rock	4000.000	400.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.486	10.531
V_x (kip)	-2.510	-4.183
V_z (kip)	-0.012	-0.018
M_x (kipft)	-0.024	-0.036
M_z (kipft)	22.213	37.508

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,

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

$$H_o = \frac{(-2.51 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.83667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(22.213 \text{ kipft}) + ((-2.51 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.4043 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6616 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.012 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.004 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.024 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.008 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.65735 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6616 \text{ ft}), (0.65735 \text{ ft})]$$

$$L_{e,req} = 5.662 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.662 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.90592$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.486 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.91758 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.91758 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.2294$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.0833$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.83667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.4043 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.4043 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.4043 \text{ kipft/ft})) + (4 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3334 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (7.4043 \text{ kipft/ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (7.4043 \text{ kipft/ft})) + (2 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.49781 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (7.4043 \text{ kipft/ft})) + ((-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 2.3113 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.3334 \text{ ft})}{2}$$

$$p_a = 0.86668 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.49781 \text{ kip/ft}^2)}{(0.86668 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.57439$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 2.5 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(2.3113 \text{ kip/ft}^2)}{(2.5 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92454$$

Status: **PASS**
Ratio: **0.570**

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = -0.004 \text{ kip/ft}$ - Lateral force per length of pile.

$M_o = 0.008 \text{ kipft/ft}$ - Overturning moment per length of pile.

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.008 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.008 \text{ kipft/ft})) + (4 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5186 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.008 \text{ kipft/ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.008 \text{ kipft/ft})) + (2 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0021446 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.008 \text{ kipft/ft})) + ((-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.0021715 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.5186 \text{ ft})}{2}$$

$$p_a = 0.90372 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0021446 \text{ kip/ft}^2)}{(0.90372 \text{ kip/ft}^2)}$$

$$(0.00012 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.0023731$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 2.5 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

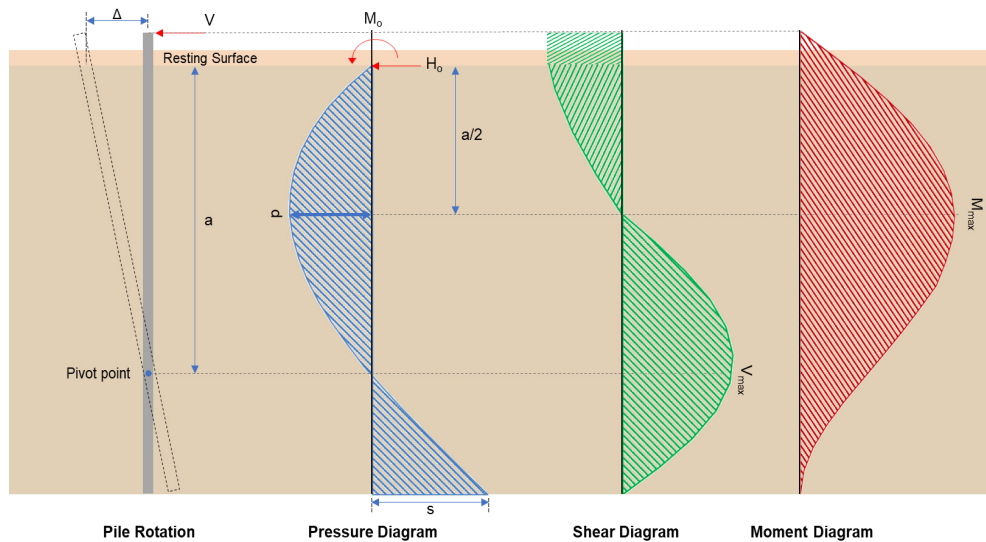
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0021715 \text{ kip/ft}^2)}{(2.5 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.00086861$$

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,

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

$$H_o = \frac{(-4.183 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3943 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(37.508 \text{ kipft}) + ((-4.183 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 12.503 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.503 \text{ kipft/ft})}{(-1.3943 \text{ kip/ft})}$$

$$E = 8.9668 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.503 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-1.3943 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (12.503 \text{ kipft/ft})) + (4 \times (-1.3943 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3319 \text{ ft}$$

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

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

$$V_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3319 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3319 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.377 \text{ kip}$$

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

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

$$M_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(8.9668 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 39.437 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.018 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.012 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.012 \text{ kipft/ft})}{(-0.006 \text{ kip/ft})}$$

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

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.006 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.012 \text{ kipft/ft})) + (4 \times (-0.006 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5186 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5186 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5186 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.022268 \text{ kip}$$

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

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

$$M_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.059609 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

Table 22.4.2.1

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.531 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.044 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.044 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.531 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0083985$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.531 \text{ kip} \rightarrow 10531 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(10531 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.226 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.226 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.226 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.226 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.357 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.377 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.377 \text{ kip})}{(74.357 \text{ kip})}$$

$$Ratio = 0.17991$$

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

Considering z-direction:

$V_{max} = 0.022268 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.022268 \text{ kip})}{(74.357 \text{ kip})}$$

$$Ratio = 0.00029947$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 39.437 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(39.437 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.6358$$

Status: **PASS**
Ratio: **0.640**

Considering z-direction:

$M_{max} = 0.059609 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.059609 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.00096102$$

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

REFERENCES	CALCULATIONS	RESULTS
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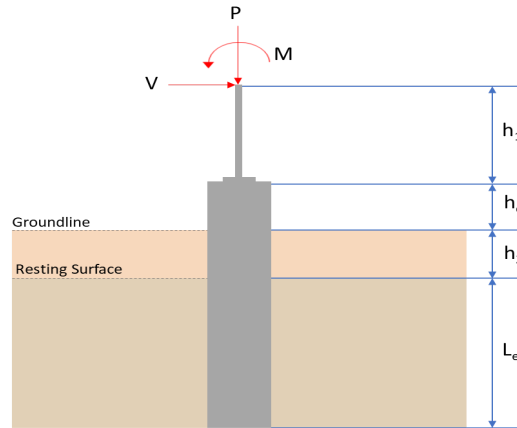
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6.25$ 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	Sedimentary & foliated rock	4000.000	400.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.486	10.531
V_x (kip)	-2.510	-4.183
V_z (kip)	0.012	0.018
M_x (kipft)	0.024	0.036
M_z (kipft)	22.213	37.508

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,

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

$$H_o = \frac{(-2.51 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.83667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(22.213 \text{ kipft}) + ((-2.51 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.4043 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6616 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.012 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.004 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.024 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.008 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.78749 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6616 \text{ ft}), (0.78749 \text{ ft})]$$

$$L_{e,req} = 5.662 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.662 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.90592$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.486 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.91758 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.91758 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.2294$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.0833$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.83667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.4043 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.4043 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.4043 \text{ kipft/ft})) + (4 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3334 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (7.4043 \text{ kipft/ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (7.4043 \text{ kipft/ft})) + (2 \times (-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.49781 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (7.4043 \text{ kipft/ft})) + ((-0.83667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 2.3113 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.3334 \text{ ft})}{2}$$

$$p_a = 0.86668 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.49781 \text{ kip/ft}^2)}{(0.86668 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.57439$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 2.5 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(2.3113 \text{ kip/ft}^2)}{(2.5 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92454$$

Status: **PASS**
Ratio: **0.570**

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = 0.004 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.008 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.008 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.008 \text{ kipft/ft})) + (4 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5186 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.008 \text{ kipft/ft})) + (3 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.008 \text{ kipft/ft})) + (2 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0046657 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.008 \text{ kipft/ft})) + ((0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0098925 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(4.5186 \text{ ft})}{2}$$

$$p_a = 0.90372 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0046657 \text{ kip/ft}^2)}{(0.90372 \text{ kip/ft}^2)}$$

$$(0.00912 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.0051628$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 2.5 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

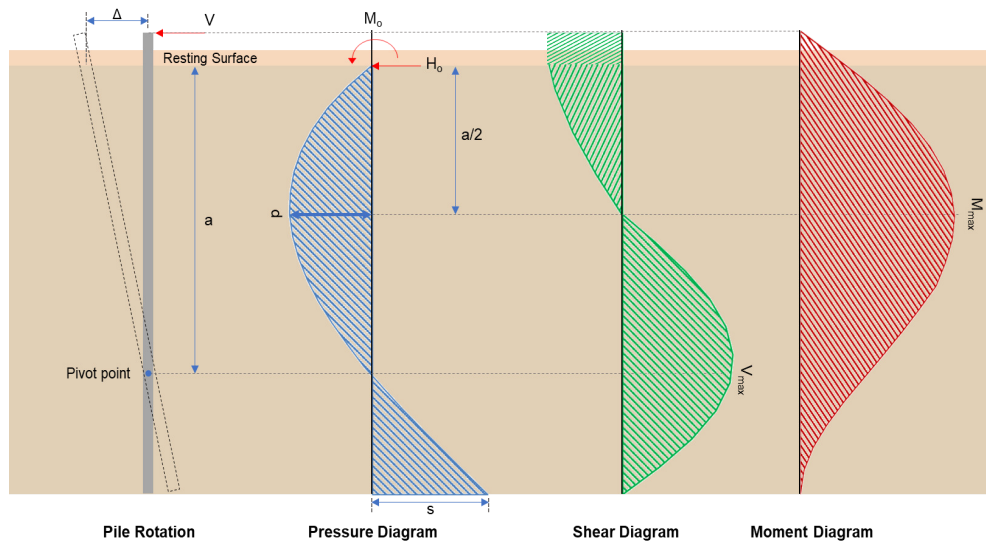
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.0098925 \text{ kip/ft}^2)}{(2.5 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.003957$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.183 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3943 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(37.508 \text{ kipft}) + ((-4.183 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 12.503 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.503 \text{ kipft/ft})}{(-1.3943 \text{ kip/ft})}$$

$$E = 8.9668 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.503 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-1.3943 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (12.503 \text{ kipft/ft})) + (4 \times (-1.3943 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3319 \text{ ft}$$

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

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

$$V_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3319 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3319 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.377 \text{ kip}$$

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

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

$$M_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(8.9668 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.9668 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3319 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 39.437 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.018 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.012 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.012 \text{ kipft/ft})}{(0.006 \text{ kip/ft})}$$

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

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.006 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.012 \text{ kipft/ft})) + (4 \times (0.006 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5186 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5186 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5186 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.022268 \text{ kip}$$

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

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

$$M_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5186 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.059609 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.531 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.044 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.044 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.531 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0083985$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.531 \text{ kip} \rightarrow 10531 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(10531 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.226 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.226 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.226 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.226 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.357 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.377 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.377 \text{ kip})}{(74.357 \text{ kip})}$$

$$Ratio = 0.17991$$

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

Considering z-direction:

$V_{max} = 0.022268 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.022268 \text{ kip})}{(74.357 \text{ kip})}$$

$$Ratio = 0.00029947$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 39.437 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(39.437 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.6358$$

Status: **PASS**
Ratio: **0.640**

Considering z-direction:

$M_{max} = 0.059609 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.059609 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.00096102$$

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