

Project Name: MTSOLAR_L6JAE9DA5J6H-St Ignatius Health
Date: Tue May 20 2025
Location: St Ignatius, MT 59865, USA
Number of Modules: 65
Unique ID: 5P-19.75-6TOP-HD-57-L-5Hx13W-91C4
Number of Poles: 5
Date Sold:
Dealer: _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	98.28 ft
Winter Tilt Angle	5
Front Edge Clearance	13 ft

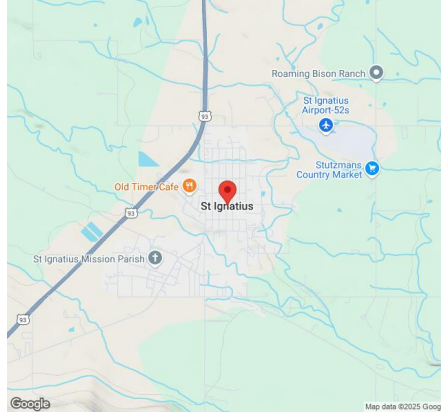
MT Solar Bill of Materials (5P-19.75-6TOP-HD-57-L-5Hx13W-91C4)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (226in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: St Ignatius, MT 59865, USA

Array Specification

Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	5
Front Edge Clearance:	13
Total Array Height at Tilt:	14.64 ft
Total Frame Length:	96.00 ft
Module Info/Notes:	
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	98.28 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	13.82 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft Pile 3: 6.50 ft Pile 4: 6.25 ft Pile 5: 6.25 ft
Foundation Volume:	8.247 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	St Ignatius, MT 59865, USA
Wind Speed:	99 mph

Snow Load:

20 psf

Design Disclaimer

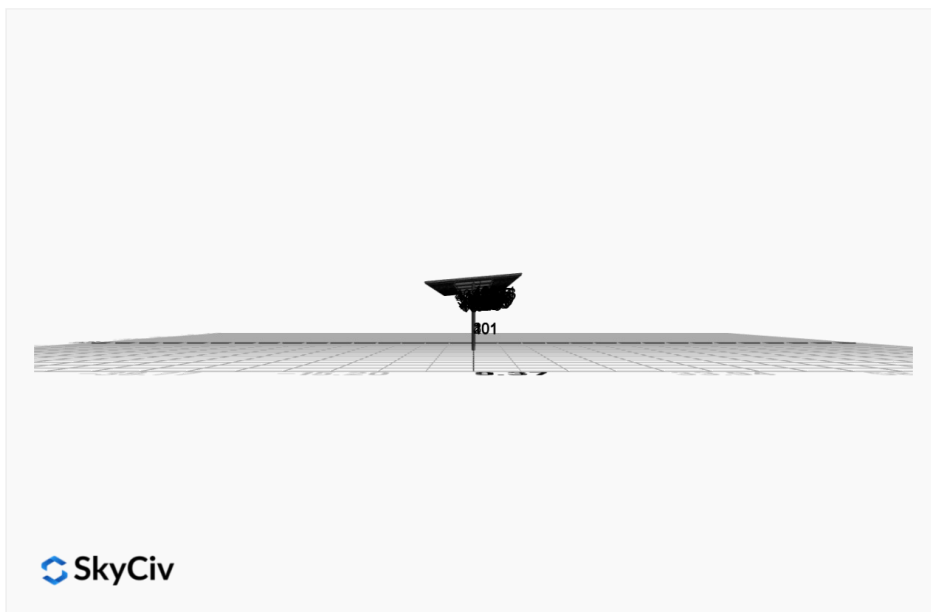
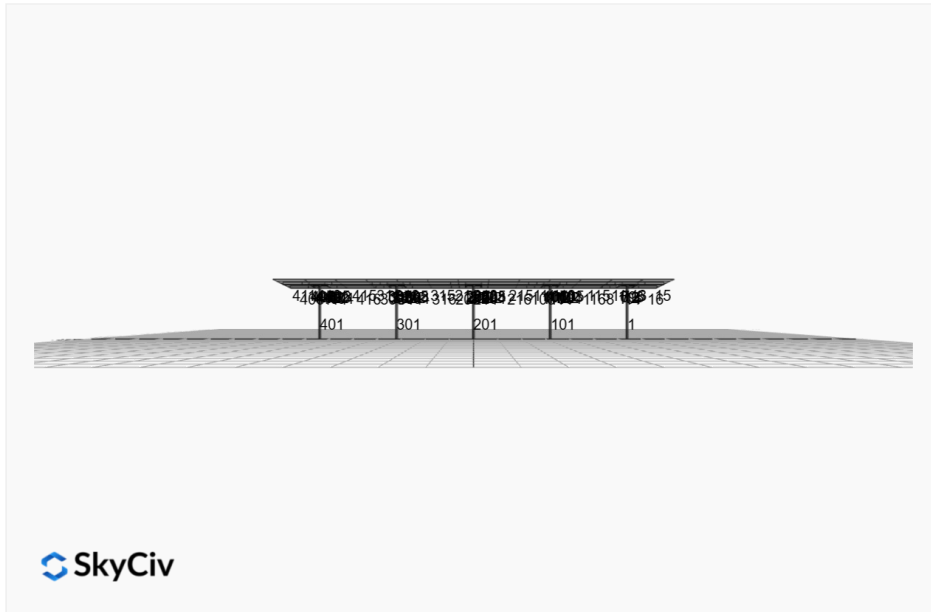
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

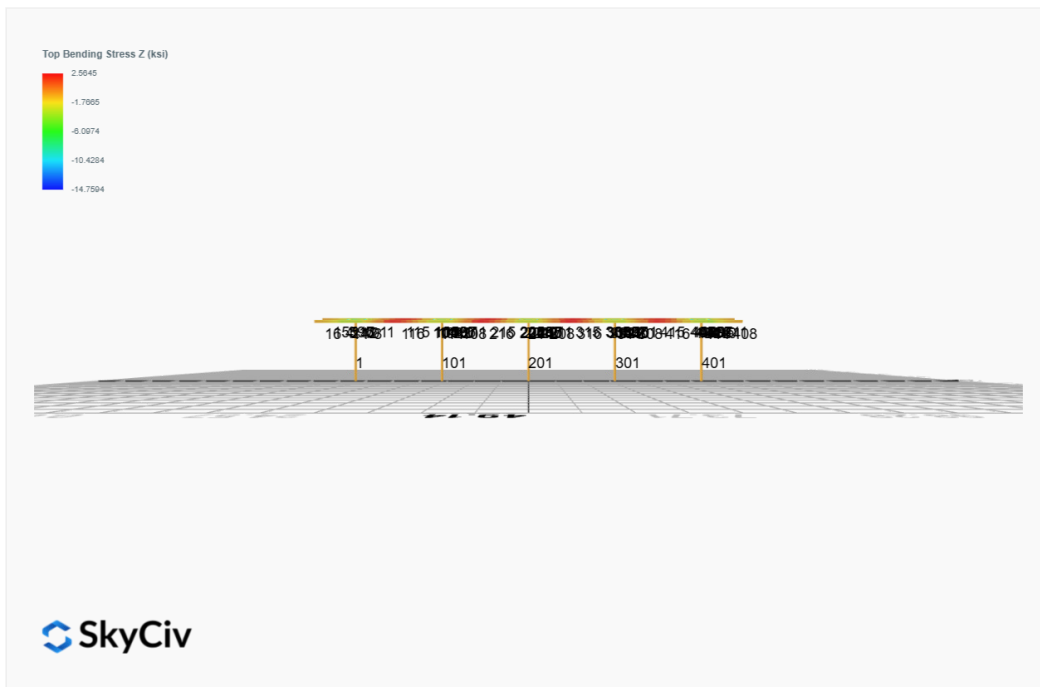
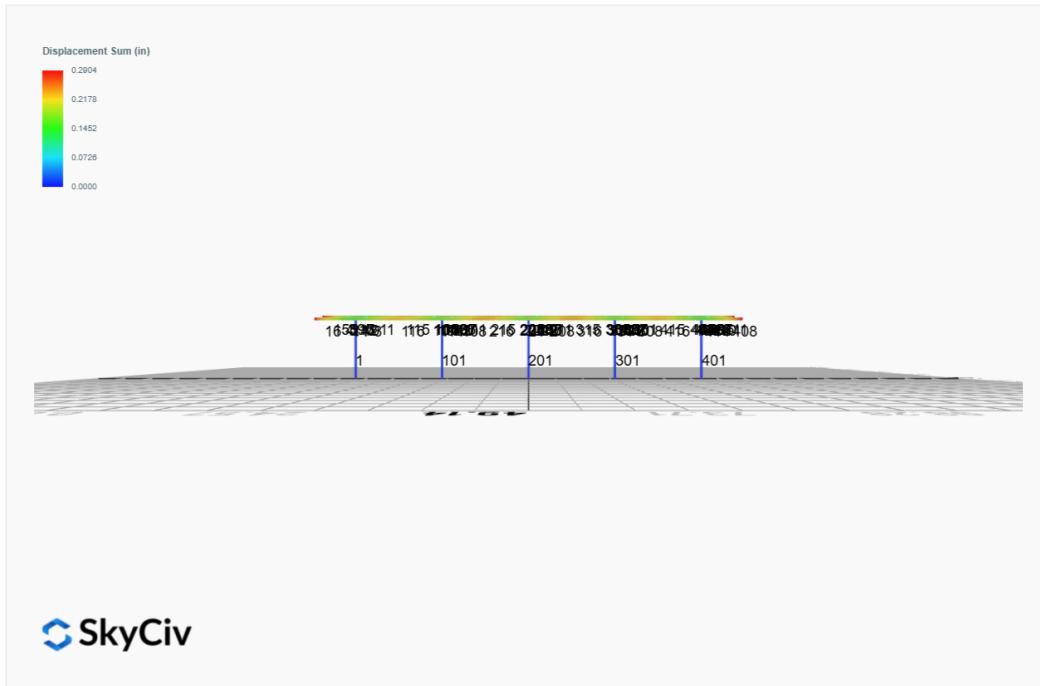
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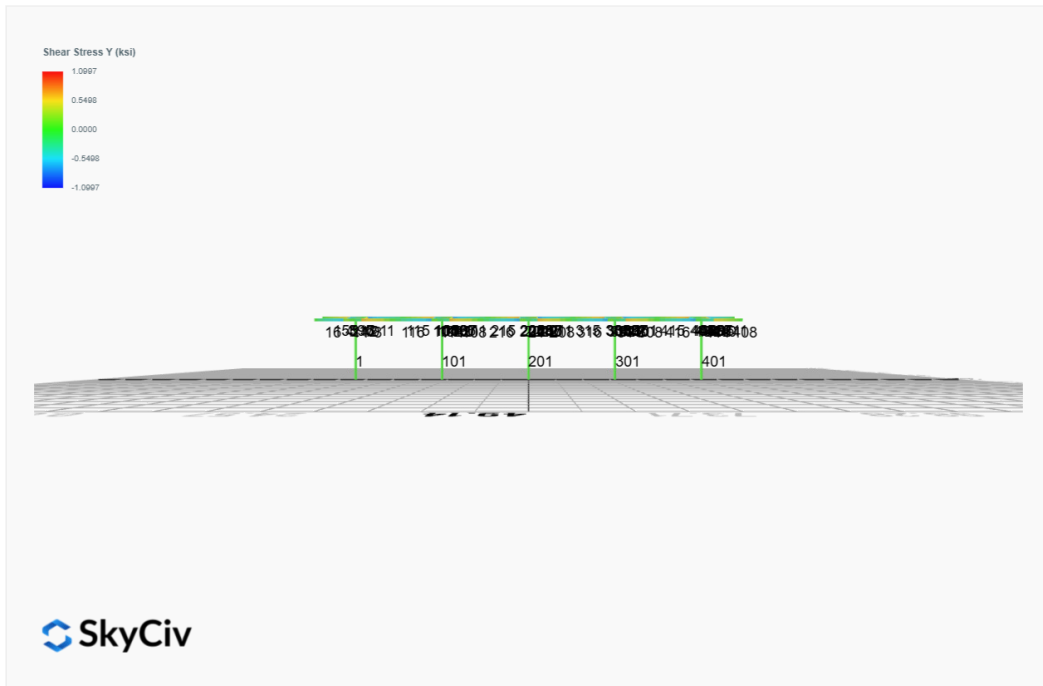
Design Notes:

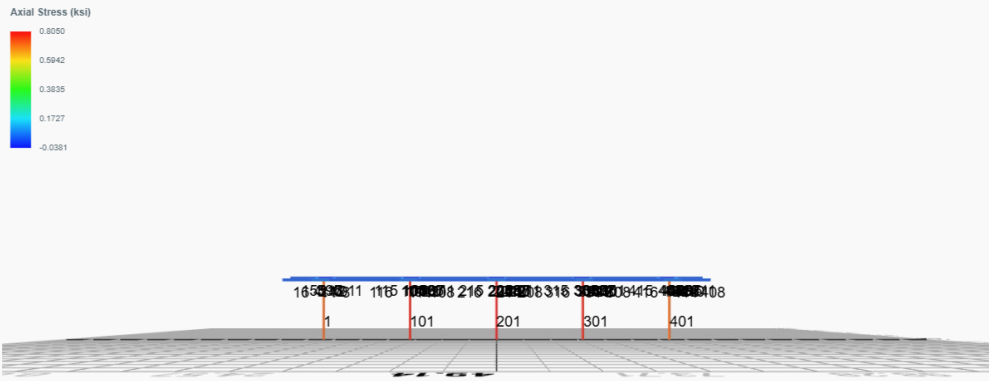
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0007	2.4212	-0.0051	-0.0224	0.0029	0.0374
ULS: 2. D + L	-0.0007	2.4212	-0.0051	-0.0224	0.0029	0.0374
ULS: 3. D + (S or Lr or R)	-0.0022	6.6121	-0.0157	-0.0700	0.0091	0.0591
ULS: 3. D + (S or Lr or R)	-0.0007	2.4212	-0.0051	-0.0224	0.0029	0.0374
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0018	5.5644	-0.0131	-0.0581	0.0075	0.0537
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0007	2.4212	-0.0051	-0.0224	0.0029	0.0374
ULS: 5b. D + 0.7E	-0.0007	2.4212	-0.0051	-0.0224	0.0029	0.0374
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0018	5.5644	-0.0131	-0.0581	0.0075	0.0537
ULS: 8. 0.6D + 0.7E	-0.0004	1.4527	-0.0030	-0.0135	0.0017	0.0225
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1867	4.5144	-0.0100	-0.0447	0.0032	3.1211
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1867	4.5144	-0.0100	-0.0447	0.0032	3.1211
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0469	1.8566	-0.0033	-0.0148	0.0017	1.4921
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1221	1.0920	-0.0026	-0.0113	0.0047	-6.2330
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1413	7.1343	-0.0168	-0.0748	0.0077	2.3665
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1413	7.1343	-0.0168	-0.0748	0.0077	2.3665
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0338	5.1409	-0.0118	-0.0524	0.0066	1.1447
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0903	4.5674	-0.0112	-0.0497	0.0089	-4.6491
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1402	3.9911	-0.0088	-0.0391	0.0031	2.3502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1402	3.9911	-0.0088	-0.0391	0.0031	2.3502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0350	1.9978	-0.0038	-0.0167	0.0020	1.1284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0914	1.4243	-0.0032	-0.0141	0.0043	-4.6654
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1864	3.5459	-0.0080	-0.0357	0.0020	3.1062
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1864	3.5459	-0.0080	-0.0357	0.0020	3.1062
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0471	0.8881	-0.0013	-0.0058	0.0005	1.4771
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1224	0.1235	-0.0006	-0.0023	0.0036	-6.2479

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	11.3552
Shear X	-0.3121
Shear Z	-0.0273
Moment X	-0.1227
Moment Y (Twist)	0.0156
Moment Z	11.2559

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	7.1343
Shear X	-0.1867
Shear Z	-0.0168
Moment X	-0.0748
Moment Y (Twist)	0.0091
Moment Z	6.2479

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	0.0009	2.5383	0.0022	0.0099	-0.0006	0.0206
ULS: 2. D + L	0.0009	2.5383	0.0022	0.0099	-0.0006	0.0206
ULS: 3. D + (S or Lr or R)	0.0027	6.9820	0.0069	0.0310	-0.0020	0.0063
ULS: 3. D + (S or Lr or R)	0.0009	2.5383	0.0022	0.0099	-0.0006	0.0206
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0022	5.8710	0.0057	0.0257	-0.0017	0.0099

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0009	2.5383	0.0022	0.0099	-0.0006	0.0206
ULS: 5b. D + 0.7E	0.0009	2.5383	0.0022	0.0099	-0.0006	0.0206
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0022	5.8710	0.0057	0.0257	-0.0017	0.0099
ULS: 8. 0.6D + 0.7E	0.0005	1.5230	0.0013	0.0059	-0.0004	0.0124
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1914	4.7568	0.0049	0.0218	-0.0027	3.2109
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1914	4.7568	0.0049	0.0218	-0.0027	3.2109
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0543	1.9395	0.0018	0.0080	-0.0014	1.4965
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1201	1.1300	-0.0000	0.0001	0.0032	-6.4034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1420	7.5349	0.0077	0.0346	-0.0032	2.4025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1420	7.5349	0.0077	0.0346	-0.0032	2.4025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0423	5.4219	0.0054	0.0243	-0.0023	1.1167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0916	4.8148	0.0040	0.0184	0.0012	-4.8082
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1433	4.2022	0.0042	0.0188	-0.0022	2.4133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1433	4.2022	0.0042	0.0188	-0.0022	2.4133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0409	2.0892	0.0019	0.0085	-0.0012	1.1275
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0903	1.4821	0.0005	0.0025	0.0022	-4.7974
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1918	3.7415	0.0040	0.0178	-0.0025	3.2026
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1918	3.7415	0.0040	0.0178	-0.0025	3.2026
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0540	0.9242	0.0009	0.0041	-0.0012	1.4882
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1197	0.1147	-0.0009	-0.0039	0.0034	-6.4117

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	12.0045
Shear X	-0.3205
Shear Z	0.0124
Moment X	0.0565
Moment Y (Twist)	0.0063
Moment Z	11.5868

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.5349
Shear X	-0.1918
Shear Z	0.0077
Moment X	0.0346
Moment Y (Twist)	0.0034
Moment Z	6.4117

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	-0.0003	2.5616	-0.0000	0.0000	-0.0000	0.0311
ULS: 2. D + L	-0.0003	2.5616	-0.0000	0.0000	-0.0000	0.0311
ULS: 3. D + (S or Lr or R)	-0.0009	7.0548	0.0000	0.0000	-0.0000	0.0391
ULS: 3. D + (S or Lr or R)	-0.0003	2.5616	-0.0000	0.0000	-0.0000	0.0311
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0007	5.9315	0.0000	0.0000	-0.0000	0.0371
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0003	2.5616	-0.0000	0.0000	-0.0000	0.0311
ULS: 5b. D + 0.7E	-0.0003	2.5616	-0.0000	0.0000	-0.0000	0.0311
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0007	5.9315	0.0000	0.0000	-0.0000	0.0371
ULS: 8. 0.6D + 0.7E	-0.0002	1.5370	-0.0000	0.0000	-0.0000	0.0187
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1946	4.8058	-0.0000	0.0000	-0.0000	3.2586
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1946	4.8058	-0.0000	0.0000	-0.0000	3.2586
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0542	1.9560	-0.0000	0.0000	-0.0000	1.5223
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1192	1.1368	-0.0000	0.0000	-0.0000	-6.4616

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1465	7.6146	0.0000	0.0000	-0.0000	2.4577
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1465	7.6146	0.0000	0.0000	-0.0000	2.4577
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0401	5.4773	0.0000	0.0000	-0.0000	1.1555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0889	4.8629	-0.0000	0.0000	-0.0000	-4.8325
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1460	4.2447	-0.0000	0.0000	-0.0000	2.4517
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1460	4.2447	-0.0000	0.0000	-0.0000	2.4517
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0406	2.1074	-0.0000	0.0000	-0.0000	1.1495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0894	1.4930	-0.0000	0.0000	-0.0000	-4.8385
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1945	3.7811	-0.0000	0.0000	-0.0000	3.2461
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1945	3.7811	-0.0000	0.0000	-0.0000	3.2461
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0544	0.9314	-0.0000	0.0000	-0.0000	1.5099
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1194	0.1121	-0.0000	0.0000	-0.0000	-6.4741

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	12.1332
Shear X	-0.3240
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	11.6936

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.6146
Shear X	-0.1946
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	6.4741

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	0.0009	2.5383	-0.0022	-0.0099	0.0006	0.0206
ULS: 2. D + L	0.0009	2.5383	-0.0022	-0.0099	0.0006	0.0206
ULS: 3. D + (S or Lr or R)	0.0027	6.9820	-0.0069	-0.0310	0.0020	0.0063
ULS: 3. D + (S or Lr or R)	0.0009	2.5383	-0.0022	-0.0099	0.0006	0.0206
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0022	5.8710	-0.0057	-0.0257	0.0017	0.0099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0009	2.5383	-0.0022	-0.0099	0.0006	0.0206
ULS: 5b. D + 0.7E	0.0009	2.5383	-0.0022	-0.0099	0.0006	0.0206
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0022	5.8710	-0.0057	-0.0257	0.0017	0.0099
ULS: 8. 0.6D + 0.7E	0.0005	1.5230	-0.0013	-0.0059	0.0004	0.0124
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1914	4.7568	-0.0049	-0.0218	0.0027	3.2109
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1914	4.7568	-0.0049	-0.0218	0.0027	3.2109
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0543	1.9395	-0.0018	-0.0080	0.0014	1.4965
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1201	1.1300	0.0000	-0.0001	-0.0032	-6.4034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1420	7.5349	-0.0077	-0.0346	0.0032	2.4025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1420	7.5349	-0.0077	-0.0346	0.0032	2.4025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0423	5.4219	-0.0054	-0.0243	0.0023	1.1167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0916	4.8148	-0.0040	-0.0184	-0.0012	-4.8082
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1433	4.2022	-0.0042	-0.0188	0.0022	2.4133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1433	4.2022	-0.0042	-0.0188	0.0022	2.4133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0409	2.0892	-0.0019	-0.0085	0.0012	1.1275
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0903	1.4821	-0.0005	-0.0025	-0.0022	-4.7974

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1918	3.7415	-0.0040	-0.0178	0.0025	3.2026
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1918	3.7415	-0.0040	-0.0178	0.0025	3.2026
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0540	0.9242	-0.0009	-0.0041	0.0012	1.4882
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1197	0.1147	0.0009	0.0039	-0.0034	-6.4117

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	12.0045
Shear X	-0.3205
Shear Z	-0.0124
Moment X	-0.0564
Moment Y (Twist)	0.0063
Moment Z	11.5869

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.5349
Shear X	-0.1918
Shear Z	-0.0077
Moment X	-0.0346
Moment Y (Twist)	0.0034
Moment Z	6.4117

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0007	2.4212	0.0051	0.0225	-0.0029	0.0374
ULS: 2. D + L	-0.0007	2.4212	0.0051	0.0225	-0.0029	0.0374
ULS: 3. D + (S or Lr or R)	-0.0022	6.6121	0.0157	0.0700	-0.0091	0.0591
ULS: 3. D + (S or Lr or R)	-0.0007	2.4212	0.0051	0.0225	-0.0029	0.0374
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0018	5.5644	0.0131	0.0581	-0.0075	0.0537
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0007	2.4212	0.0051	0.0225	-0.0029	0.0374
ULS: 5b. D + 0.7E	-0.0007	2.4212	0.0051	0.0225	-0.0029	0.0374
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0018	5.5644	0.0131	0.0581	-0.0075	0.0537
ULS: 8. 0.6D + 0.7E	-0.0004	1.4527	0.0030	0.0135	-0.0017	0.0225
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1867	4.5144	0.0100	0.0447	-0.0032	3.1211
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1867	4.5144	0.0100	0.0447	-0.0032	3.1211
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0469	1.8566	0.0033	0.0148	-0.0017	1.4921
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1221	1.0920	0.0026	0.0113	-0.0047	-6.2330
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1413	7.1343	0.0168	0.0748	-0.0077	2.3665
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1413	7.1343	0.0168	0.0748	-0.0077	2.3665
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0338	5.1409	0.0118	0.0524	-0.0066	1.1447
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0903	4.5674	0.0112	0.0497	-0.0089	-4.6491
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1402	3.9911	0.0088	0.0391	-0.0031	2.3502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1402	3.9911	0.0088	0.0391	-0.0031	2.3502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0350	1.9978	0.0038	0.0167	-0.0020	1.1284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0914	1.4243	0.0032	0.0141	-0.0043	-4.6654
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1864	3.5459	0.0080	0.0357	-0.0020	3.1062
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1864	3.5459	0.0080	0.0357	-0.0020	3.1062
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0471	0.8881	0.0013	0.0058	-0.0005	1.4771
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1224	0.1235	0.0006	0.0023	-0.0036	-6.2479

Worst Case Reactions LRFD

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

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	11.3552
Shear X	-0.3121
Shear Z	0.0273
Moment X	0.1227
Moment Y (Twist)	0.0156
Moment Z	11.2560

Result	Value (kip, kip-ft)
Axial	7.1343
Shear X	-0.1867
Shear Z	0.0168
Moment X	0.0748
Moment Y (Twist)	0.0091
Moment Z	6.2479

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States

 User Name: sales@mtsolar.us
 Project Name: MTSOLAR_L6JAE9DA5J6H-St Ignatius Health
 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				

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

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	LS T	LS C	L D	
1	7	29.02	29.02	13.82	-	300	200	1	
2	5	1.30	1.30	2.00	-	300	200	1	
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.30,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.11 9,1.26,1.18,1.18,1.19,1.17	300	200	1	
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.68,1.67,1.67,1.74,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.6 9,1.68,1.67,1.67,1.37,1.68	300	200	1	
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.84,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.6 8,1.78,1.67,1.67,1.68,1.66	300	200	1	
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.38,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.11 9,1.27,1.18,1.18,1.19,1.17	300	200	1	
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,2.32,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6 8,1.81,1.67,1.67,1.68,1.66	300	200	1	
8	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.11,2.11,2.11,2.29,2.11,2.11,2.06,2.37,2.12,2.12,2.12,2.13,2.11,2.11,2.1 1,2.27,2.11,2.11,1.88,2.34	300	200	1	
9	2	2.60	2.60	4.00	-	300	200	1	
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.68,1.67,1.67,1.75,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.6 9,1.68,1.67,1.67,1.43,1.68	300	200	1	
11	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.17,1.01,2.12,2.12,2.24,2.35,2.12,2.12,2.12,2.11,2.12,2.12,2.1 6,1.37,2.13,2.13,2.30,2.37	300	200	1	
12	5	1.30	1.30	2.00	-	300	200	1	
13	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.10,1.10,1.09,1.09,1.10,1.14,1.10,1.10,1.10,1.10,1.09,1.09,1.1 0,1.10,1.09,1.09,1.10,1.11	300	200	1	
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.11,1.09,1.09,1.09,1.09,1.09,1.0 9,1.10,1.09,1.09,3.05,1.14	300	200	1	
15	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3 3,2.33,2.33,2.33,2.33,2.33	300	200	1	
16	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3 3,2.33,2.33,2.33,2.33,2.33	300	200	1	
101	7	29.02	29.02	13.82	-	300	200	1	
102	5	1.30	1.30	2.00	-	300	200	1	
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.29,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.11 9,1.25,1.18,1.18,1.19,1.17	300	200	1	
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.67,1.67,1.74,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.6 9,1.68,1.67,1.67,1.39,1.68	300	200	1	
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.82,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6 8,1.77,1.67,1.67,1.68,1.66	300	200	1	
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.30,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.11 9,1.25,1.18,1.18,1.19,1.17	300	200	1	
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.91,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6 8,1.78,1.67,1.67,1.68,1.66	300	200	1	
108	19	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.05,2.07,2.07,2.07,2.07,2.07,2.0 7,2.07,2.07,2.07,2.34,2.07	300	200	1	
109	2	2.60	2.60	4.00	-	300	200	1	
110	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.67,1.67,1.74,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.6 9,1.68,1.67,1.67,1.42,1.68	300	200	1	
111	19	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.28,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.0 7,2.08,2.07,2.07,2.07,2.07	300	200	1	
112	5	1.30	1.30	2.00	-	300	200	1	

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	23.27	6.12	40.24	43.62
114	133.20	85.85	23.54	6.12	40.24	43.62
115	133.20	69.16	17.59	6.12	40.24	43.62
116	133.20	69.16	17.60	6.12	40.24	43.62
201	251.16	52.42	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	23.07	6.12	40.24	43.62
214	133.20	85.85	23.52	6.12	40.24	43.62
215	133.20	69.16	17.81	6.12	40.24	43.62
216	133.20	69.16	17.83	6.12	40.24	43.62
301	251.16	52.42	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	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.28	6.12	40.24	43.62
314	133.20	85.85	23.54	6.12	40.24	43.62
315	133.20	69.16	17.79	6.12	40.24	43.62
316	133.20	69.16	17.64	6.12	40.24	43.62
401	251.16	52.42	42.30	42.30	75.35	75.35
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28

407	116.10	114.23	15.79	11.10	42.08	23.28
408	133.20	32.95	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	32.95	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	25.08	6.12	40.24	43.62
414	133.20	85.85	24.88	6.12	40.24	43.62
415	133.20	69.16	17.16	6.12	40.24	43.62
416	133.20	69.16	17.71	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	0.217	0.266	0.006	0.004	0.000	0.293	#16	0.775	Not Required	Pass
2	0.001	0.454	0.018	0.094	0.003	0.472	#21	0.035	Not Required	Pass
3	0.002	0.655	0.010	0.066	0.001	0.666	#21	0.045	Not Required	Pass
4	0.002	0.644	0.029	0.065	0.006	0.661	#21	0.080	Not Required	Pass
5	0.002	0.406	0.031	0.065	0.008	0.414	#21	0.074	Not Required	Pass
6	0.002	0.645	0.010	0.065	0.001	0.656	#21	0.045	Not Required	Pass
7	0.002	0.400	0.033	0.065	0.008	0.408	#21	0.074	Not Required	Pass
8	0.000	0.048	0.031	0.045	0.003	0.072	#21	0.095	Not Required	Pass
9	0.003	0.076	0.008	0.001	0.000	0.086	#21	0.204	Not Required	Pass
10	0.002	0.633	0.033	0.064	0.007	0.656	#21	0.080	Not Required	Pass
11	0.000	0.048	0.031	0.045	0.003	0.073	#21	0.095	Not Required	Pass
12	0.001	0.442	0.018	0.092	0.003	0.460	#21	0.035	Not Required	Pass
13	0.001	0.261	0.075	0.058	0.004	0.320	#21	0.286	Not Required	Pass
14	0.002	0.261	0.075	0.057	0.004	0.317	#21	0.190	Not Required	Pass
15	0.000	0.101	0.039	0.035	0.002	0.140	#21	Not Required	Not Required	Pass
16	0.000	0.099	0.039	0.034	0.002	0.138	#21	Not Required	Not Required	Pass
101	0.229	0.274	0.003	0.004	0.000	0.302	#16	0.775	Not Required	Pass
102	0.001	0.467	0.018	0.098	0.003	0.485	#21	0.035	Not Required	Pass
103	0.002	0.686	0.011	0.069	0.002	0.698	#21	0.045	Not Required	Pass
104	0.002	0.674	0.030	0.068	0.006	0.695	#21	0.080	Not Required	Pass
105	0.002	0.425	0.030	0.069	0.007	0.433	#21	0.074	Not Required	Pass
106	0.002	0.692	0.012	0.070	0.002	0.704	#21	0.045	Not Required	Pass
107	0.002	0.429	0.031	0.069	0.008	0.437	#21	0.074	Not Required	Pass
108	0.000	0.058	0.027	0.044	0.003	0.069	#21	0.095	Not Required	Pass
109	0.002	0.074	0.008	0.001	0.000	0.083	#21	0.204	Not Required	Pass
110	0.002	0.679	0.029	0.068	0.006	0.698	#21	0.080	Not Required	Pass
111	0.000	0.059	0.028	0.044	0.003	0.069	#21	0.095	Not Required	Pass
112	0.000	0.473	0.019	0.099	0.003	0.490	#21	0.035	Not Required	Pass
113	0.001	0.216	0.071	0.057	0.004	0.267	#21	0.286	Not Required	Pass
114	0.001	0.217	0.070	0.056	0.004	0.263	#21	0.286	Not Required	Pass
115	0.000	0.232	0.039	0.044	0.003	0.271	#21	0.473	Not Required	Pass
116	0.000	0.230	0.039	0.043	0.003	0.269	#21	0.473	Not Required	Pass
201	0.231	0.276	0.000	0.004	0.000	0.305	#16	0.775	Not Required	Pass
202	0.001	0.476	0.019	0.099	0.003	0.494	#21	0.035	Not Required	Pass
203	0.002	0.696	0.011	0.070	0.002	0.708	#21	0.045	Not Required	Pass
204	0.002	0.684	0.030	0.069	0.006	0.705	#21	0.080	Not Required	Pass
205	0.002	0.433	0.031	0.070	0.008	0.440	#21	0.074	Not Required	Pass

203	0.002	0.432	0.031	0.070	0.006	0.440	#21	0.074	Not Required	Pass
206	0.002	0.696	0.011	0.070	0.002	0.708	#21	0.045	Not Required	Pass
207	0.002	0.432	0.031	0.070	0.008	0.440	#21	0.074	Not Required	Pass
208	0.000	0.056	0.027	0.044	0.003	0.065	#21	0.095	Not Required	Pass
209	0.002	0.074	0.008	0.001	0.000	0.083	#21	0.204	Not Required	Pass
210	0.002	0.684	0.030	0.069	0.006	0.705	#21	0.080	Not Required	Pass
211	0.000	0.056	0.028	0.045	0.003	0.065	#21	0.095	Not Required	Pass
212	0.001	0.476	0.019	0.099	0.003	0.494	#21	0.035	Not Required	Pass
213	0.001	0.223	0.071	0.058	0.004	0.274	#21	0.286	Not Required	Pass
214	0.002	0.225	0.070	0.057	0.004	0.271	#21	0.286	Not Required	Pass
215	0.001	0.241	0.039	0.045	0.003	0.280	#21	0.473	Not Required	Pass
216	0.000	0.238	0.039	0.044	0.003	0.277	#21	0.473	Not Required	Pass
301	0.229	0.274	0.003	0.004	0.000	0.302	#16	0.775	Not Required	Pass
302	0.000	0.473	0.019	0.099	0.003	0.490	#21	0.035	Not Required	Pass
303	0.002	0.692	0.012	0.070	0.002	0.704	#21	0.045	Not Required	Pass
304	0.002	0.679	0.029	0.068	0.006	0.698	#21	0.080	Not Required	Pass
305	0.002	0.429	0.031	0.069	0.008	0.437	#21	0.074	Not Required	Pass
306	0.002	0.686	0.011	0.069	0.002	0.698	#21	0.045	Not Required	Pass
307	0.002	0.425	0.030	0.069	0.007	0.433	#21	0.074	Not Required	Pass
308	0.000	0.056	0.027	0.043	0.003	0.066	#21	0.095	Not Required	Pass
309	0.002	0.074	0.008	0.001	0.000	0.083	#21	0.204	Not Required	Pass
310	0.002	0.674	0.030	0.068	0.006	0.695	#21	0.080	Not Required	Pass
311	0.000	0.056	0.027	0.044	0.003	0.066	#21	0.095	Not Required	Pass
312	0.001	0.467	0.018	0.098	0.003	0.485	#21	0.035	Not Required	Pass
313	0.001	0.216	0.071	0.057	0.004	0.267	#21	0.286	Not Required	Pass
314	0.001	0.217	0.070	0.056	0.004	0.263	#21	0.286	Not Required	Pass
315	0.001	0.242	0.039	0.044	0.003	0.281	#21	0.473	Not Required	Pass
316	0.000	0.240	0.039	0.044	0.003	0.278	#21	0.473	Not Required	Pass
401	0.217	0.266	0.006	0.004	0.000	0.293	#16	0.775	Not Required	Pass
402	0.001	0.442	0.018	0.092	0.003	0.460	#21	0.035	Not Required	Pass
403	0.002	0.645	0.010	0.065	0.001	0.656	#21	0.045	Not Required	Pass
404	0.002	0.633	0.033	0.064	0.007	0.656	#21	0.080	Not Required	Pass
405	0.002	0.400	0.033	0.065	0.008	0.408	#21	0.074	Not Required	Pass
406	0.002	0.655	0.010	0.066	0.001	0.666	#21	0.045	Not Required	Pass
407	0.002	0.406	0.031	0.065	0.008	0.414	#21	0.074	Not Required	Pass
408	0.000	0.099	0.039	0.034	0.002	0.138	#21	Not Required	Not Required	Pass
409	0.003	0.076	0.008	0.001	0.000	0.086	#21	0.204	Not Required	Pass
410	0.002	0.644	0.029	0.065	0.006	0.661	#21	0.080	Not Required	Pass
411	0.000	0.101	0.039	0.035	0.002	0.140	#21	Not Required	Not Required	Pass
412	0.001	0.454	0.018	0.094	0.003	0.472	#21	0.035	Not Required	Pass
413	0.001	0.261	0.075	0.058	0.004	0.320	#21	0.190	Not Required	Pass
414	0.002	0.261	0.075	0.057	0.004	0.317	#21	0.286	Not Required	Pass
415	0.000	0.229	0.039	0.045	0.003	0.268	#21	0.473	Not Required	Pass
416	0.000	0.227	0.039	0.045	0.003	0.266	#21	0.473	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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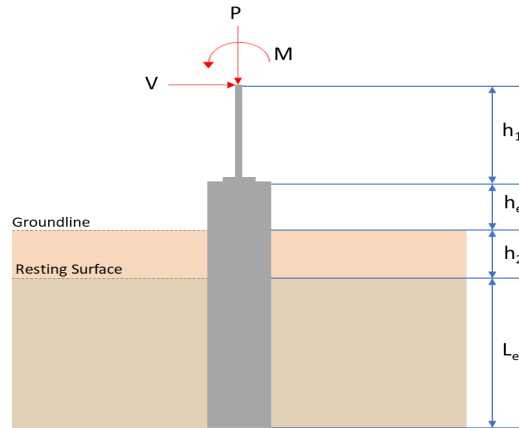
SkyCiv Foundation Design

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

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.134	11.355
V_x (kip)	-0.187	-0.312
V_z (kip)	-0.017	-0.027
M_x (kipft)	-0.075	-0.123
M_z (kipft)	6.248	11.256

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,

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

$$H_o = \frac{(-0.187 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.062333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.248 \text{ kipft}) + ((-0.187 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.0827 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.0906 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.017 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0056667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075 \text{ kipft}) + ((-0.017 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.025 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3433 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.0906 \text{ ft}), (1.3433 \text{ ft})]$$

$$L_{e,req} = 6.091 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.091 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.97456$$

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

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

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

$$q = \frac{(7.134 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0093 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.0093 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.50463$$

Status: **PASS**
Ratio: **0.500**

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:

$H_o = -0.062333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.0827 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0827 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (2.0827 \text{ kipft/ft})) + (4 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2244 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.0827 \text{ kipft/ft})) + (3 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (2.0827 \text{ kipft/ft})) + (2 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.28285 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.0827 \text{ kipft/ft})) + ((-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91101 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2244 \text{ ft})}{2}$$

$$p_a = 0.31683 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28285 \text{ kip/ft}^2)}{(0.31683 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89273$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.91101 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97175$$

Status: **PASS**
Ratio: **0.890**

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

Considering z-direction:

$H_o = -0.0056667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.025 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.025 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.025 \text{ kipft/ft})) + (4 \times (-0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4196 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.025 \text{ kipft/ft})) + (3 \times (-0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.025 \text{ kipft/ft})) + (2 \times (-0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.00028272 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.025 \text{ kipft/ft})) + ((-0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0035187 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4196 \text{ ft})}{2}$$

$$p_a = 0.33147 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00028272 \text{ kip/ft}^2)}{(0.33147 \text{ kip/ft}^2)}$$

(0.0035187 kip/ft²)

$$Ratio = 0.00085292$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

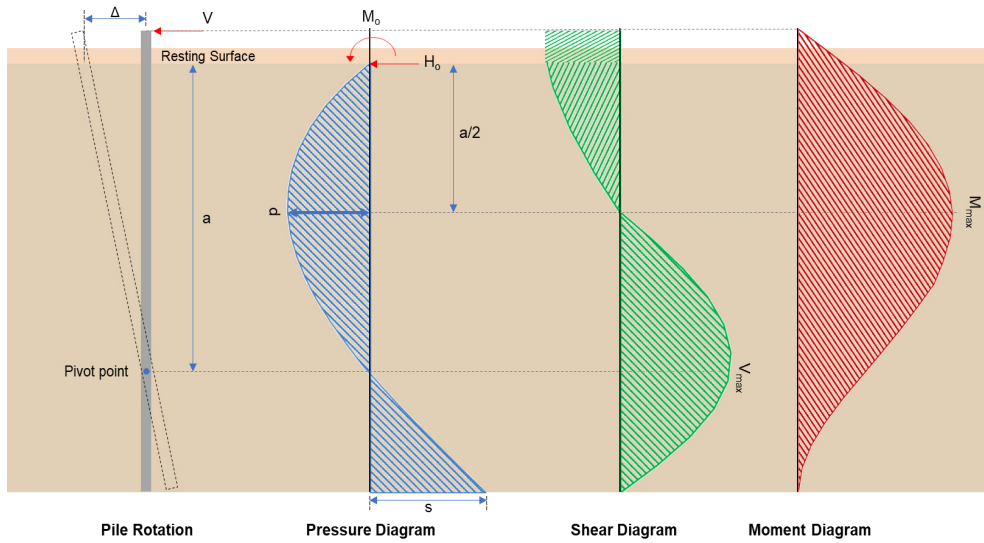
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0035187 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0037532$$

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,

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

$$H_o = \frac{(-0.312 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.104 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.256 \text{ kipft}) + ((-0.312 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.752 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.752 \text{ kipft/ft})}{(-0.104 \text{ kip/ft})}$$

$$E = 36.077 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.104 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.752 \text{ kipft/ft})) + (4 \times (-0.104 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2206 \text{ ft}$$

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

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

$$V_{max} = ((-0.104 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2206 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2206 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4 \text{ kip}$$

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

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

$$M_{max} = ((-0.104 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(36.077 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.446 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.027 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.009 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((-0.027 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.041 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.041 \text{ kipft/ft})}{(-0.009 \text{ kip/ft})}$$

$$E = 4.5556 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.041 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.009 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.041 \text{ kipft/ft})) + (4 \times (-0.009 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4155 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.009 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.052717 \text{ kip}$$

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

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

$$M_{max} = ((-0.009 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.5556 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.14961 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.355 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.018 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.018 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(11.355 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0090557$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.355 \text{ kip} \rightarrow 11355 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(11355 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.366 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.366 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.366 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.366 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.448 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.4 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.4 \text{ kip})}{(74.448 \text{ kip})}$$

$$Ratio = 0.045669$$

Considering z-direction:

$V_{max} = 0.052717 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.052717 \text{ kip})}{(74.448 \text{ kip})}$$

$$Ratio = 0.00070811$$

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

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:

$M_{max} = 10.446 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.446 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.16841$$

Status: **PASS**
 Ratio: **0.170**

Considering z-direction:

$M_{max} = 0.14961 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.14961 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.002412$$

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

REFERENCES	CALCULATIONS	RESULTS
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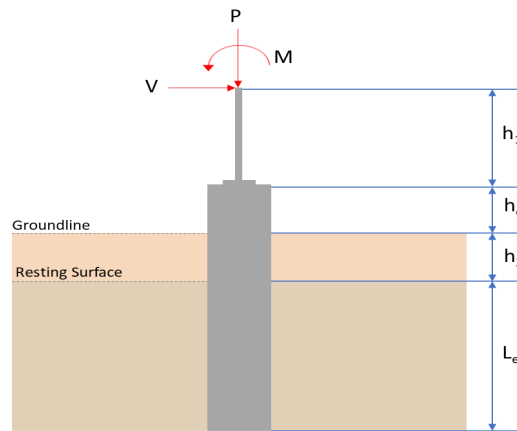
SkyCiv Foundation Design

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	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.535	12.004
V_x (kip)	-0.192	-0.320
V_z (kip)	0.008	0.012
M_x (kipft)	0.035	0.056
M_z (kipft)	6.412	11.587

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,

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

$$H_o = \frac{(-0.192 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.412 \text{ kipft}) + ((-0.192 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.1373 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.1406 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.008 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0026667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((0.008 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.011667 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2099 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.1406 \text{ ft}), (1.2099 \text{ ft})]$$

$$L_{e,req} = 6.141 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.141 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.98256$$

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

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

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

$$q = \frac{(7.535 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.066 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.066 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.53299$$

Status: **PASS**
Ratio: **0.530**

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:

$H_o = -0.064 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.1373 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1373 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (2.1373 \text{ kipft/ft})) + (4 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2244 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.1373 \text{ kipft/ft})) + (3 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (2.1373 \text{ kipft/ft})) + (2 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.29024 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.1373 \text{ kipft/ft})) + ((-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.93488 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2244 \text{ ft})}{2}$$

$$p_a = 0.31683 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29024 \text{ kip/ft}^2)}{(0.31683 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91608$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.93488 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9972$$

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

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = 0.0026667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.011667 \text{ kipft/ft})) + (4 \times (0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4207 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011667 \text{ kipft/ft})) + (3 \times (0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.011667 \text{ kipft/ft})) + (2 \times (0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0041239 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.011667 \text{ kipft/ft})) + ((0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0096512 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4207 \text{ ft})}{2}$$

$$p_a = 0.33155 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0041239 \text{ kip/ft}^2)}{(0.33155 \text{ kip/ft}^2)}$$

$$(0.00109 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.012438$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

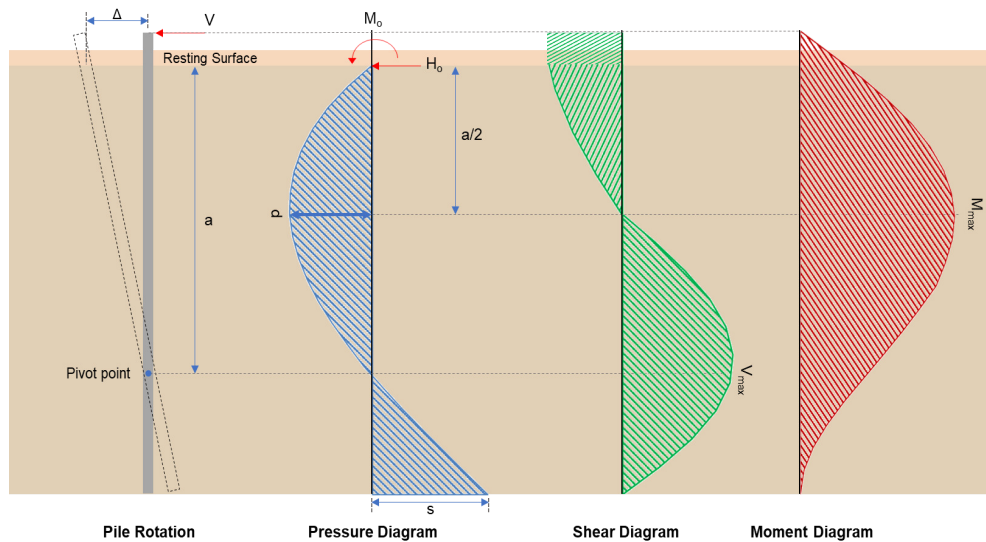
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.0096512 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.010295$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.32 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.10667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.587 \text{ kipft}) + ((-0.32 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.8623 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.8623 \text{ kipft/ft})}{(-0.10667 \text{ kip/ft})}$$

$$E = 36.209 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8623 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.10667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8623 \text{ kipft/ft})) + (4 \times (-0.10667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2204 \text{ ft}$$

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

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

$$V_{max} = ((-0.10667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2204 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2204 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4992 \text{ kip}$$

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

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

$$M_{max} = ((-0.10667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(36.209 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.751 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.056 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.018667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018667 \text{ kipft/ft})}{(0.004 \text{ kip/ft})}$$

$$E = 4.6667 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.018667 \text{ kipft/ft})) + (4 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4123 \text{ ft}$$

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

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

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

$$V_{max} = ((0.004 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4123 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4123 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023805 \text{ kip}$$

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

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

$$M_{max} = ((0.004 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.6667 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.067663 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.004 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.998 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.998 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

$$(1.8322 \text{ in}^2)$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</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;">$Ratio = \frac{(12.004 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0095733$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.004 \text{ kip} \rightarrow 12004 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(12004 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.476 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.476 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.476 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.476 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.52 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.4992 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(3.4992 \text{ kip})}{(74.52 \text{ kip})}$$

$$Ratio = 0.046956$$

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

Considering z-direction:

$V_{max} = 0.023805 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.023805 \text{ kip})}{(74.52 \text{ kip})}$$

$$Ratio = 0.00031945$$

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:

$M_{max} = 10.751 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.751 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.17333$$

Status: **PASS**
Ratio: **0.170**

Considering z-direction:

$M_{max} = 0.067663 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.067663 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0010909$$

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

REFERENCES	CALCULATIONS	RESULTS
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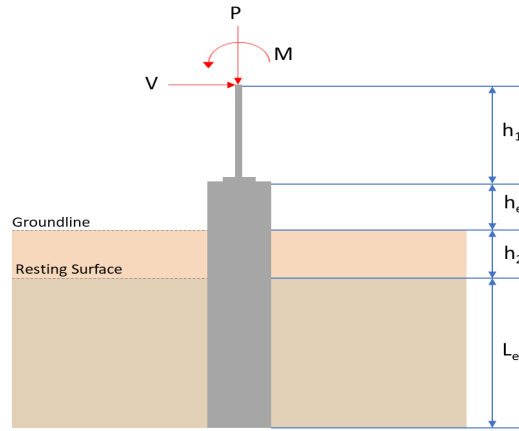
SkyCiv Foundation Design

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	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.535	12.004
V_x (kip)	-0.192	-0.320
V_z (kip)	-0.008	-0.012
M_x (kipft)	-0.035	-0.056
M_z (kipft)	6.412	11.587

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,

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

$$H_o = \frac{(-0.192 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.412 \text{ kipft}) + ((-0.192 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.1373 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.1406 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.008 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0026667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((-0.008 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.011667 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0624 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.1406 \text{ ft}), (1.0624 \text{ ft})]$$

$$L_{e,req} = 6.141 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.141 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.98256$$

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

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

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

$$q = \frac{(7.535 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.066 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.066 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.53299$$

Status: **PASS**
Ratio: **0.530**

Czeraniak

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:

$H_o = -0.064 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.1373 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1373 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (2.1373 \text{ kipft/ft})) + (4 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2244 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.1373 \text{ kipft/ft})) + (3 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (2.1373 \text{ kipft/ft})) + (2 \times (-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.29024 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.1373 \text{ kipft/ft})) + ((-0.064 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.93488 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2244 \text{ ft})}{2}$$

$$p_a = 0.31683 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29024 \text{ kip/ft}^2)}{(0.31683 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91608$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.93488 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9972$$

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

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = -0.0026667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.011667 \text{ kipft/ft})) + (4 \times (-0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4207 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011667 \text{ kipft/ft})) + (3 \times (-0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.011667 \text{ kipft/ft})) + (2 \times (-0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.00020105 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.011667 \text{ kipft/ft})) + ((-0.0026667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0016085 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4207 \text{ ft})}{2}$$

$$p_a = 0.33155 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00020105 \text{ kip/ft}^2)}{(0.33155 \text{ kip/ft}^2)}$$

$$(0.00160 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.00060637$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

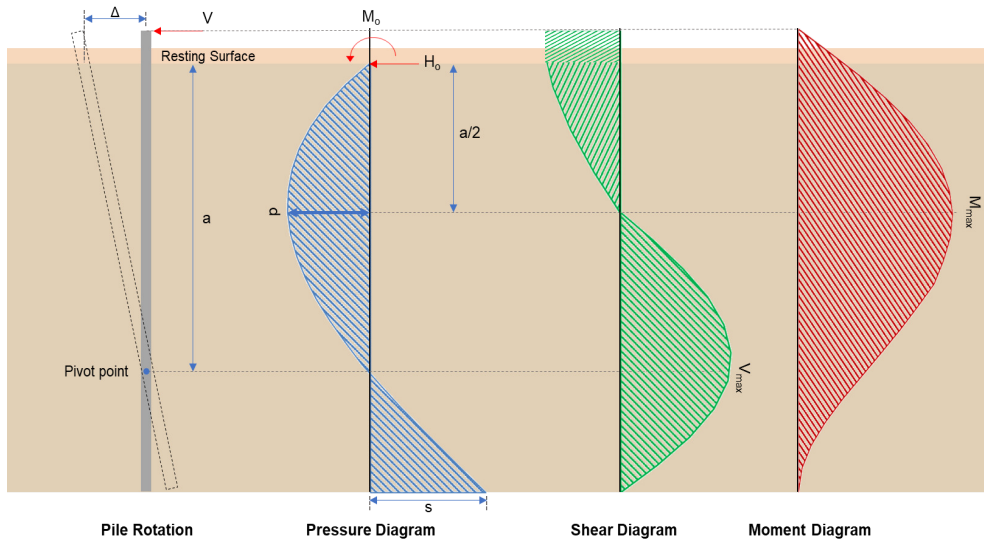
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.0016085 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0017158$$

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,

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

$$H_o = \frac{(-0.32 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.10667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.587 \text{ kipft}) + ((-0.32 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.8623 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.8623 \text{ kipft/ft})}{(-0.10667 \text{ kip/ft})}$$

$$E = 36.209 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8623 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.10667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8623 \text{ kipft/ft})) + (4 \times (-0.10667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2204 \text{ ft}$$

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

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

$$V_{max} = ((-0.10667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2204 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2204 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4992 \text{ kip}$$

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

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

$$M_{max} = ((-0.10667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(36.209 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (36.209 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2204 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.751 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.056 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.018667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018667 \text{ kipft/ft})}{(-0.004 \text{ kip/ft})}$$

$$E = 4.6667 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.018667 \text{ kipft/ft})) + (4 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4123 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.004 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4123 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4123 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023805 \text{ kip}$$

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

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

$$M_{max} = ((-0.004 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.6667 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.6667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4123 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.067663 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.004 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.998 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.998 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(12.004 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0095733$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.004 \text{ kip} \rightarrow 12004 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(12004 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.476 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.476 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.476 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.476 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.52 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.4992 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.4992 \text{ kip})}{(74.52 \text{ kip})}$$

$$Ratio = 0.046956$$

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

Considering z-direction:

$V_{max} = 0.023805 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.023805 \text{ kip})}{(74.52 \text{ kip})}$$

$$Ratio = 0.00031945$$

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:

$M_{max} = 10.751 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.751 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.17333$$

Status: **PASS**
Ratio: **0.170**

Considering z-direction:

$M_{max} = 0.067663 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.067663 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0010909$$

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

REFERENCES	CALCULATIONS	RESULTS
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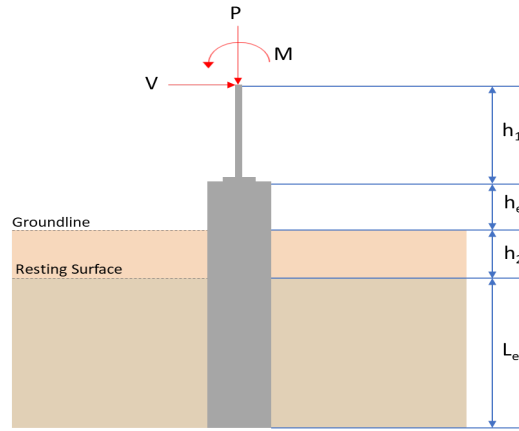
SkyCiv Foundation Design

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	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.134	11.355
V_x (kip)	-0.187	-0.312
V_z (kip)	0.017	0.027
M_x (kipft)	0.075	0.123
M_z (kipft)	6.248	11.256

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,

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

$$H_o = \frac{(-0.187 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.062333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.248 \text{ kipft}) + ((-0.187 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.0827 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.0906 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.017 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0056667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075 \text{ kipft}) + ((0.017 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.025 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.586 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.0906 \text{ ft}), (1.586 \text{ ft})]$$

$$L_{e,req} = 6.091 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.091 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.97456$$

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

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

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

$$q = \frac{(7.134 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0093 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.0093 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.50463$$

Status: **PASS**
Ratio: **0.500**

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:

$H_o = -0.062333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.0827 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0827 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (2.0827 \text{ kipft/ft})) + (4 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2244 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.0827 \text{ kipft/ft})) + (3 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (2.0827 \text{ kipft/ft})) + (2 \times (-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.28285 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.0827 \text{ kipft/ft})) + ((-0.062333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91101 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2244 \text{ ft})}{2}$$

$$p_a = 0.31683 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28285 \text{ kip/ft}^2)}{(0.31683 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89273$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.91101 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97175$$

Status: **PASS**
Ratio: **0.890**

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

Considering z-direction:

$H_o = 0.0056667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.025 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.025 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.025 \text{ kipft/ft})) + (4 \times (0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4196 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.025 \text{ kipft/ft})) + (3 \times (0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.025 \text{ kipft/ft})) + (2 \times (0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0087966 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.025 \text{ kipft/ft})) + ((0.0056667 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.020609 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4196 \text{ ft})}{2}$$

$$p_a = 0.33147 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0087966 \text{ kip/ft}^2)}{(0.33147 \text{ kip/ft}^2)}$$

(0.026538)

$$Ratio = 0.026538$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

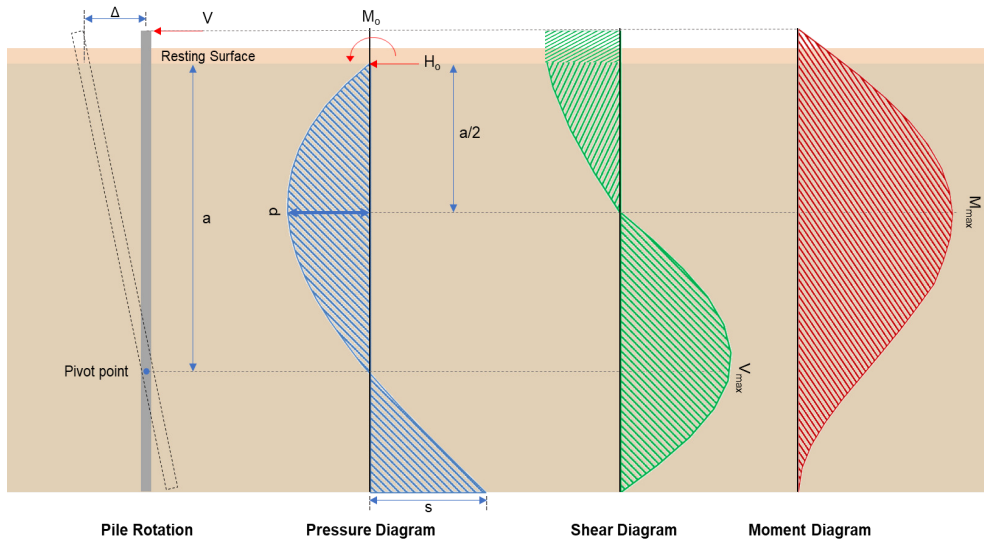
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.020609 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.021983$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.312 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.104 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.256 \text{ kipft}) + ((-0.312 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.752 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.752 \text{ kipft/ft})}{(-0.104 \text{ kip/ft})}$$

$$E = 36.077 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.104 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.752 \text{ kipft/ft})) + (4 \times (-0.104 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2206 \text{ ft}$$

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

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

$$V_{max} = ((-0.104 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2206 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2206 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4 \text{ kip}$$

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

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

$$M_{max} = ((-0.104 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(36.077 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (36.077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2206 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.446 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.027 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.009 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((0.027 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.041 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.041 \text{ kipft/ft})}{(0.009 \text{ kip/ft})}$$

$$E = 4.5556 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.041 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.009 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.041 \text{ kipft/ft})) + (4 \times (0.009 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4155 \text{ ft}$$

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

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

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

$$V_{max} = ((0.009 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.052717 \text{ kip}$$

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

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

$$M_{max} = ((0.009 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.5556 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.5556 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.14961 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.355 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.018 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.018 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

$$(1.8322 \text{ in}^2)$$

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;">$Ratio = \frac{(11.355 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0090557$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.355 \text{ kip} \rightarrow 11355 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(11355 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.366 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.366 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.366 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.366 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.448 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.4 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.4 \text{ kip})}{(74.448 \text{ kip})}$$

$$Ratio = 0.045669$$

Considering z-direction:

$V_{max} = 0.052717 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.052717 \text{ kip})}{(74.448 \text{ kip})}$$

$$Ratio = 0.00070811$$

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

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:

$M_{max} = 10.446 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.446 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.16841$$

Status: **PASS**
 Ratio: **0.170**

Considering z-direction:

$M_{max} = 0.14961 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.14961 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.002412$$

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

REFERENCES	CALCULATIONS	RESULTS
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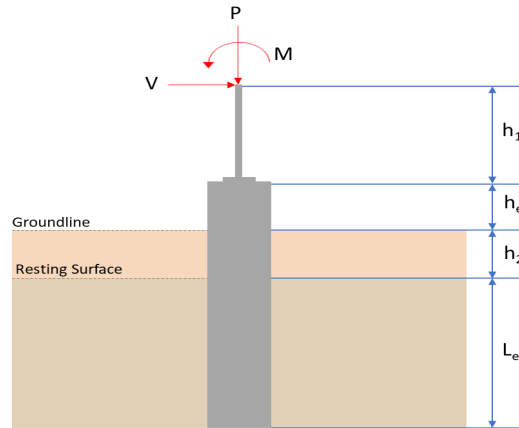
SkyCiv Foundation Design

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

$L = 6.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.615	12.133
V_x (kip)	-0.195	-0.324
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	6.474	11.694

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,

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

$$H_o = \frac{(-0.195 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.065 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.474 \text{ kipft}) + ((-0.195 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.158 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.1575 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.1575 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.157 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(6.157 \text{ ft})}{(6.5 \text{ ft})}$$

$$Ratio = 0.94723$$

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

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

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

$$q = \frac{(7.615 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0773 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(1.0773 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.53865$$

Status: **PASS**
Ratio: **0.540**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.1667$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.065 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.158 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.158 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.065 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (2.158 \text{ kipft/ft})) + (4 \times (-0.065 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3959 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.158 \text{ kipft/ft})) + (3 \times (-0.065 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (2.158 \text{ kipft/ft})) + (2 \times (-0.065 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.26864 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.158 \text{ kipft/ft})) + ((-0.065 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.86855 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3959 \text{ ft})}{2}$$

$$p_a = 0.32969 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26864 \text{ kip/ft}^2)}{(0.32969 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.81483$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

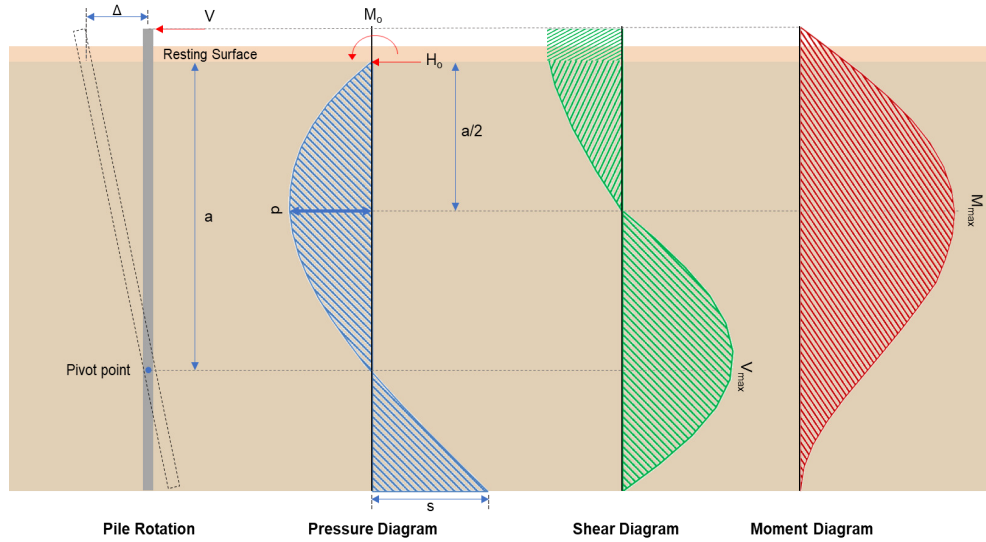
$$\dots (0.86855 \text{ kip/ft}^2)$$

Status: **PASS**
Ratio: **0.810**

$$\text{Ratio} = \frac{\dots}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89082$$

Status: **PASS**
Ratio: **0.890**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.324 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.694 \text{ kipft}) + ((-0.324 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.898 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.898 \text{ kipft/ft})}{(-0.108 \text{ kip/ft})}$$

$$E = 36.093 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.898 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.108 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.898 \text{ kipft/ft})) + (4 \times (-0.108 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3914 \text{ ft}$$

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

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

$$V_{max} = ((-0.108 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (36.093 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.3914 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (36.093 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.3914 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4043 \text{ kip}$$

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

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

$$M_{max} = ((-0.108 \text{ kip/ft}) \times (36 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(36.093 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.3914 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (36.093 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.3914 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (36.093 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.3914 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 10.87 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.133 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.994 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.994 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

Status: **PASS**
Ratio: **1.000**

<p>25.7.2.2 25.7.2.1</p>	<p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties: Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in) s_{ties} - Maximum center-to-center spacing of ties,</p> <p style="text-align: center;">$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), D]$</p> <p style="text-align: center;">$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</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>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;">$Ratio = \frac{(12.133 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0096761$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters: $b_w = 36 \text{ in}$ - 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> <p style="text-align: center;">$V_{c,max} = 186.09 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.133 \text{ kip} \rightarrow 12133 \text{ lbf}$,</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> <p style="text-align: center;">$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$</p>	

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(12133 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.498 \text{ kip}$$

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

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.498 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.498 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((76.498 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.534 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.4043 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(3.4043 \text{ kip})}{(74.534 \text{ kip})}$$

$$\text{Ratio} = 0.045674$$

Status: **PASS**
 Ratio: **0.045674**

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 $\phi M_{n,2}$

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

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

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

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: $M_{max} = 10.87 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(10.87 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.17525$$