

Your Project Calculations



Project Name: Macedonia Baptist Church 5x16

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Macedonia%20Baptist%20Church%205x16&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=Ijm4hlnNnAqmaWWnKgy6i7kFjyOtX8rj3rhaAkjrgTHNTZsliZVSzDyuHqwu32yg

Array Specification

Product:	Beam
Unique ID:	5P-19.75-6TOP-SD-24-L-5Hx16W-2IE2
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	67.00in
Number of Rows:	5
Number of Columns:	16
Total Number of Modules:	80
Desired Tilt Angle:	7
Front Edge Clearance:	8
Total Array Height at Tilt:	10.04 ft
Total Frame Length:	90.50 ft
Frame Weight:	3249 lbs
Array Dimensions N/S:	16.88 ft
Array Dimensions E/W:	90.67 ft
Rail Length:	202.50 in
Rail Spacing:	2.79 ft
Rail Check:	Not Checked

Support Specifications

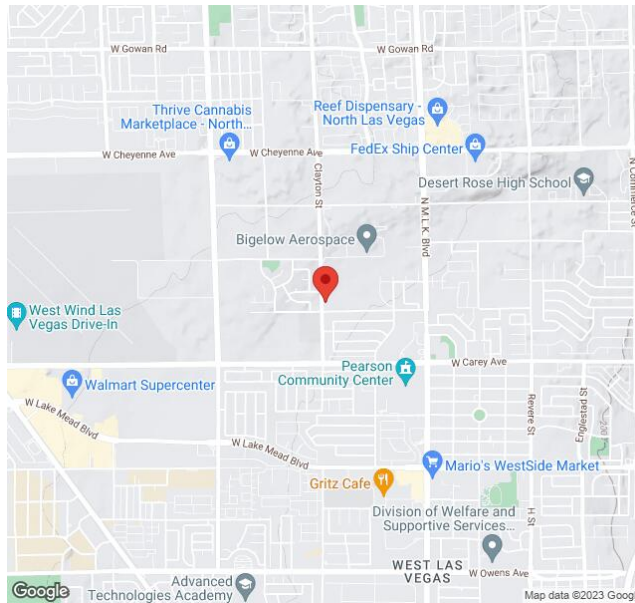
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.03 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.25 ft Pile 2: 4.50 ft Pile 3: 4.75 ft Pile 4: 4.50 ft Pile 5: 4.25 ft
Foundation Volume:	13.185 y ³
Foundation Result:	PASSED
Mount Twist:	0.152263 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2600 Clayton St, North Las Vegas, NV 89032, USA
Wind Speed:	92 mph
Snow Load:	5 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.003024 ksf



Design Disclaimer

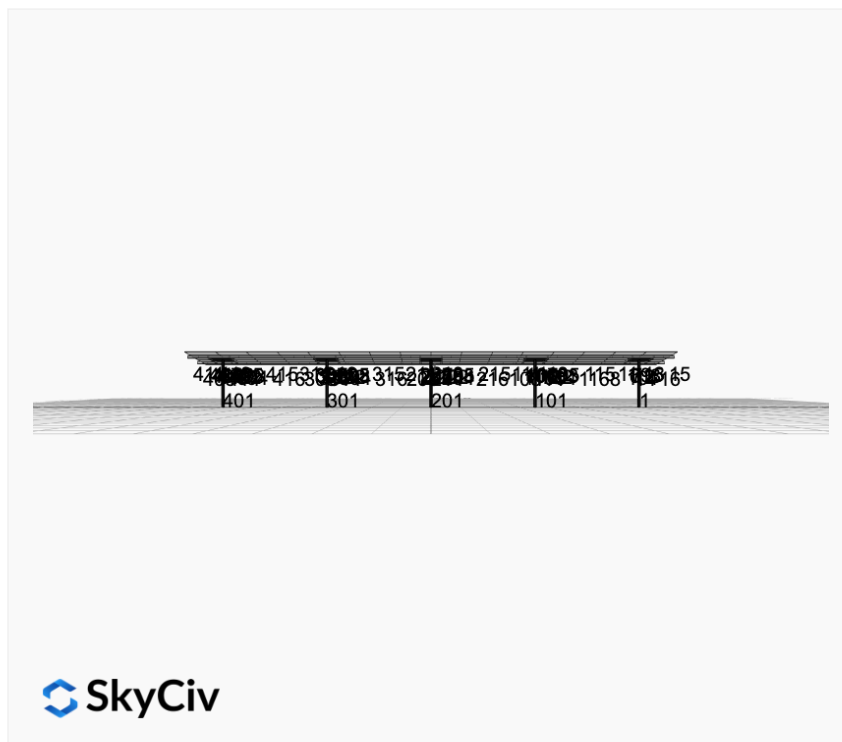
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

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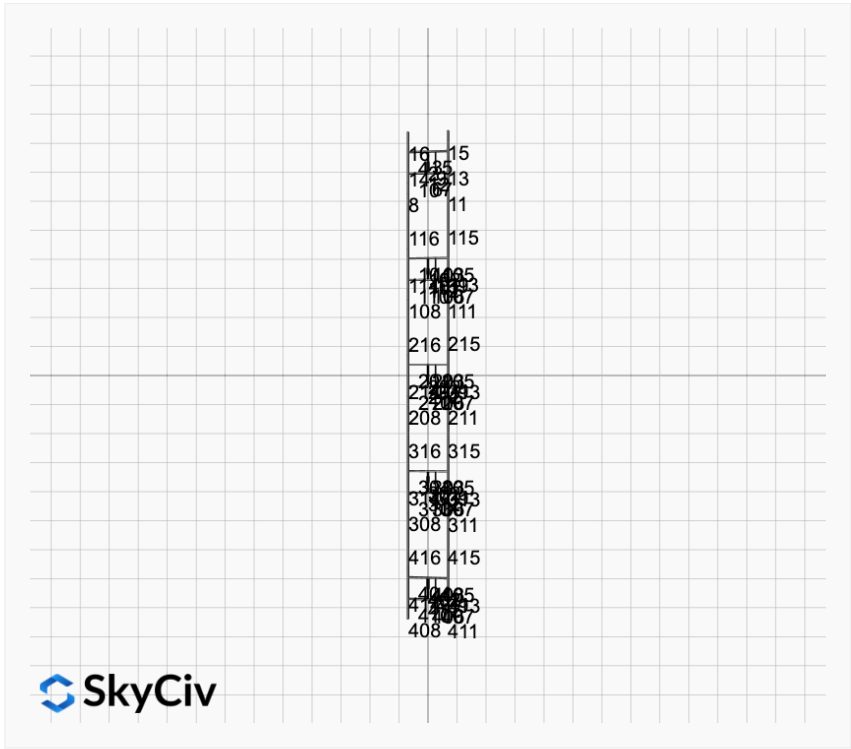
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

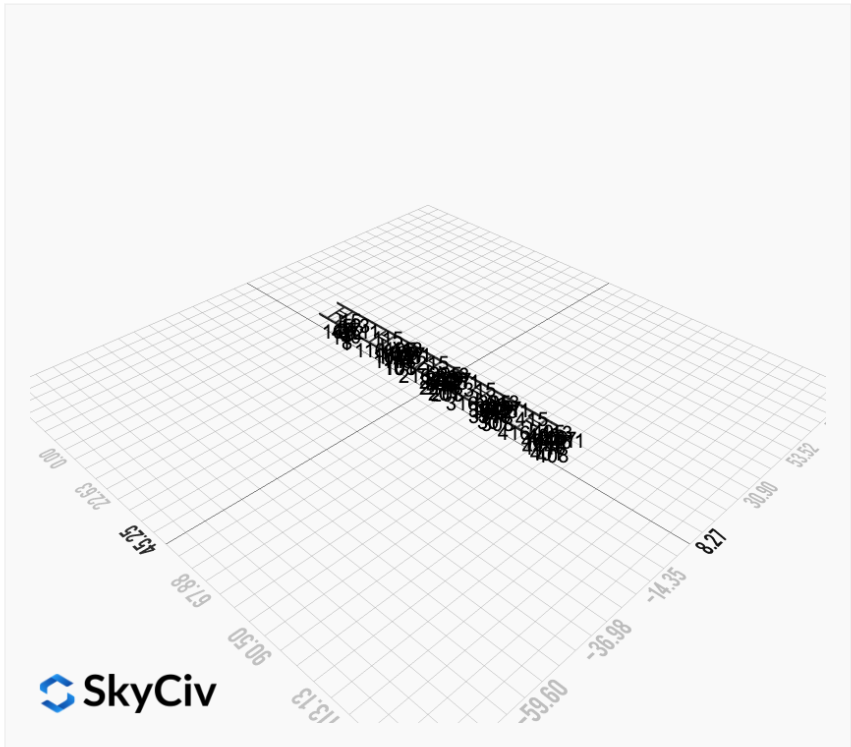


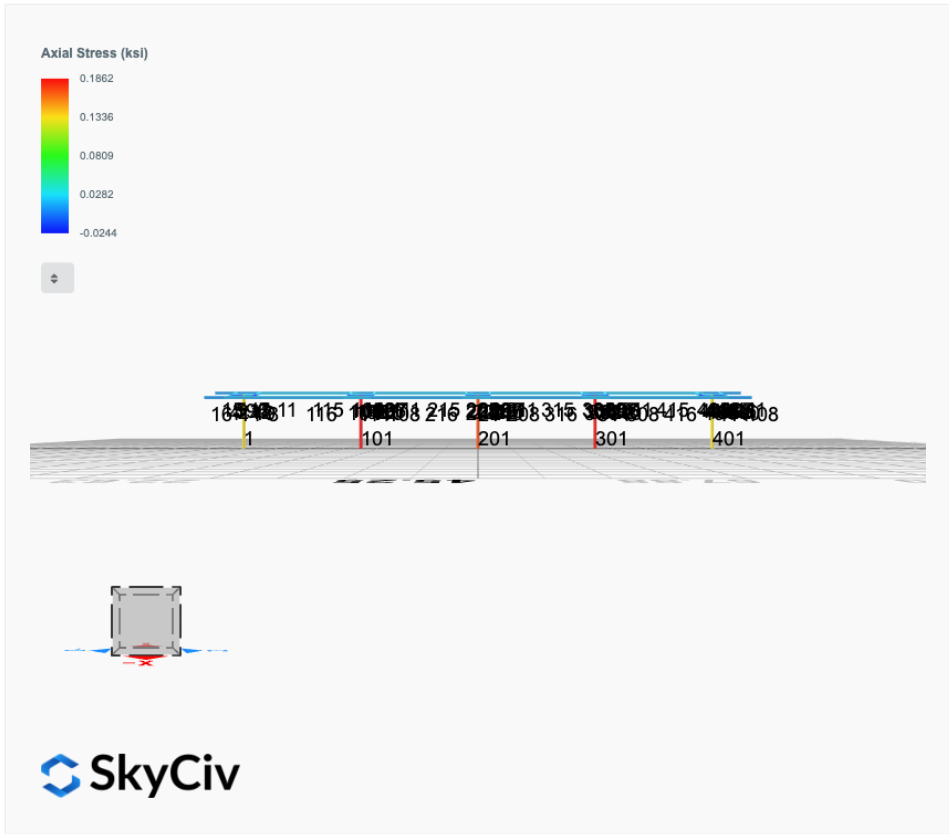


 SkyCiv



 SkyCiv





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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0067	1.7636	0.1090	0.2842	-0.0205	-0.0337
ULS: 2. D + L	0.0067	1.7636	0.1090	0.2842	-0.0205	-0.0337
ULS: 3. D + (S or Lr or R)	0.0102	2.5217	0.1664	0.4335	-0.0313	-0.0623
ULS: 3. D + (S or Lr or R)	0.0067	1.7636	0.1090	0.2842	-0.0205	-0.0337
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0093	2.3322	0.1520	0.3962	-0.0286	-0.0551
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0067	1.7636	0.1090	0.2842	-0.0205	-0.0337
ULS: 5b. D + 0.7E	0.0067	1.7636	0.1090	0.2842	-0.0205	-0.0337
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0093	2.3322	0.1520	0.3962	-0.0286	-0.0551
ULS: 8. 0.6D + 0.7E	0.0040	1.0582	0.0654	0.1705	-0.0123	-0.0202
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2743	3.9355	0.2781	0.7208	-0.0933	4.4412
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2743	3.9355	0.2781	0.7208	-0.0933	4.4412
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1635	0.4715	0.0122	0.0337	0.0138	0.0974
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1874	0.4752	0.0050	0.0172	0.0344	-6.5696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2014	3.9611	0.2788	0.7236	-0.0832	3.3010
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2014	3.9611	0.2788	0.7236	-0.0832	3.3010
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1269	1.3631	0.0794	0.2084	-0.0029	0.0432
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1448	1.3659	0.0740	0.1959	0.0126	-4.9571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2041	3.3925	0.2358	0.6116	-0.0751	3.3225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2041	3.3925	0.2358	0.6116	-0.0751	3.3225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1243	0.7945	0.0364	0.0964	0.0052	0.0646
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1422	0.7973	0.0310	0.0839	0.0207	-4.9356
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2770	3.2301	0.2344	0.6071	-0.0851	4.4546
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2770	3.2301	0.2344	0.6071	-0.0851	4.4546
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1608	-0.2340	-0.0314	-0.0799	0.0220	0.1109
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1847	-0.2302	-0.0386	-0.0965	0.0426	-6.5562

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	6.1150
Shear X	-0.4683
Shear Z	0.4425
Moment X	1.1481
Moment Y (Twist)	0.1522
Moment Z	11.1415

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	3.9611
Shear X	-0.2770
Shear Z	0.2788
Moment X	0.7236
Moment Y (Twist)	0.0933
Moment Z	6.5696

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	-0.0075	2.2992	-0.0168	-0.0453	0.0059	0.0800
ULS: 2. D + L	-0.0075	2.2992	-0.0168	-0.0453	0.0059	0.0800
ULS: 3. D + (S or Lr or R)	-0.0115	3.3386	-0.0256	-0.0692	0.0090	0.1109
ULS: 3. D + (S or Lr or R)	-0.0075	2.2992	-0.0168	-0.0453	0.0059	0.0800
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0105	3.0788	-0.0234	-0.0632	0.0083	0.1032
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0075	2.2992	-0.0168	-0.0453	0.0059	0.0800
ULS: 5b. D + 0.7E	-0.0075	2.2992	-0.0168	-0.0453	0.0059	0.0800

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0105	3.0788	-0.0234	-0.0632	0.0083	0.1032
ULS: 8. 0.6D + 0.7E	-0.0045	1.3795	-0.0101	-0.0272	0.0036	0.0480
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3595	5.2776	-0.0407	-0.1114	0.0035	5.7002
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3595	5.2776	-0.0407	-0.1114	0.0035	5.7002
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2116	0.5239	-0.0011	-0.0034	-0.0012	0.1189
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1882	0.5370	-0.0044	-0.0098	0.0188	-7.8847
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2745	5.3126	-0.0413	-0.1128	0.0065	4.3183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2745	5.3126	-0.0413	-0.1128	0.0065	4.3183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1539	1.7473	-0.0116	-0.0318	0.0029	0.1324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1363	1.7571	-0.0141	-0.0366	0.0179	-5.8703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2715	4.5330	-0.0347	-0.0949	0.0041	4.2951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2715	4.5330	-0.0347	-0.0949	0.0041	4.2951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1568	0.9677	-0.0050	-0.0139	0.0005	0.1092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1393	0.9776	-0.0075	-0.0187	0.0156	-5.8936
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3565	4.3579	-0.0340	-0.0933	0.0012	5.6682
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3565	4.3579	-0.0340	-0.0933	0.0012	5.6682
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2146	-0.3958	0.0056	0.0147	-0.0036	0.0869
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1912	-0.3827	0.0023	0.0083	0.0164	-7.9167

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.2429
Shear X	-0.5970
Shear Z	-0.0646
Moment X	-0.1773
Moment Y (Twist)	0.0310
Moment Z	13.4430

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.3126
Shear X	-0.3595
Shear Z	-0.0413
Moment X	-0.1128
Moment Y (Twist)	0.0188
Moment Z	7.9167

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	0.0016	2.2030	-0.0000	0.0000	0.0000	0.0158
ULS: 2. D + L	0.0016	2.2030	-0.0000	0.0000	0.0000	0.0158
ULS: 3. D + (S or Lr or R)	0.0025	3.1919	-0.0000	-0.0000	0.0000	0.0131
ULS: 3. D + (S or Lr or R)	0.0016	2.2030	-0.0000	0.0000	0.0000	0.0158
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0023	2.9446	-0.0000	-0.0000	0.0000	0.0138
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0016	2.2030	-0.0000	0.0000	0.0000	0.0158
ULS: 5b. D + 0.7E	0.0016	2.2030	-0.0000	0.0000	0.0000	0.0158
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0023	2.9446	-0.0000	-0.0000	0.0000	0.0138
ULS: 8. 0.6D + 0.7E	0.0010	1.3218	-0.0000	0.0000	0.0000	0.0095
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3452	5.0381	-0.0000	-0.0000	0.0000	5.6577
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3452	5.0381	-0.0000	-0.0000	0.0000	5.6577
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2101	0.5167	-0.0000	-0.0000	0.0000	0.1325
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2045	0.5203	-0.0000	-0.0000	0.0000	-8.0890
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2579	5.0710	-0.0000	-0.0000	0.0000	4.2452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2579	5.0710	-0.0000	-0.0000	0.0000	4.2452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1586	1.6799	-0.0000	-0.0000	0.0000	0.1013
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1545	1.6826	-0.0000	-0.0000	0.0000	-6.0648

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	-0.2585	4.3294	-0.0000	-0.0000	0.0000	4.2472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2585	4.3294	-0.0000	-0.0000	0.0000	4.2472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1580	0.9383	-0.0000	-0.0000	0.0000	0.1033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1538	0.9410	-0.0000	-0.0000	0.0000	-6.0628
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3459	4.1569	-0.0000	-0.0000	0.0000	5.6513
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3459	4.1569	-0.0000	-0.0000	0.0000	5.6513
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2094	-0.3645	-0.0000	-0.0000	0.0000	0.1262
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2039	-0.3609	-0.0000	-0.0000	0.0000	-8.0953

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	7.8632
Shear X	-0.5781
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	13.7768

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.0710
Shear X	-0.3459
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	8.0953

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	-0.0075	2.2992	0.0168	0.0453	-0.0059	0.0800
ULS: 2. D + L	-0.0075	2.2992	0.0168	0.0453	-0.0059	0.0800
ULS: 3. D + (S or Lr or R)	-0.0115	3.3386	0.0256	0.0692	-0.0090	0.1109
ULS: 3. D + (S or Lr or R)	-0.0075	2.2992	0.0168	0.0453	-0.0059	0.0800
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0105	3.0788	0.0234	0.0632	-0.0083	0.1032
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0075	2.2992	0.0168	0.0453	-0.0059	0.0800
ULS: 5b. D + 0.7E	-0.0075	2.2992	0.0168	0.0453	-0.0059	0.0800
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0105	3.0788	0.0234	0.0632	-0.0083	0.1032
ULS: 8. 0.6D + 0.7E	-0.0045	1.3795	0.0101	0.0272	-0.0036	0.0480
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3595	5.2776	0.0407	0.1114	-0.0035	5.7002
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3595	5.2776	0.0407	0.1114	-0.0035	5.7002
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2116	0.5239	0.0011	0.0034	0.0013	0.1189
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1882	0.5370	0.0044	0.0098	-0.0188	-7.8847
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2745	5.3126	0.0413	0.1128	-0.0065	4.3183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2745	5.3126	0.0413	0.1128	-0.0065	4.3183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1539	1.7473	0.0116	0.0318	-0.0029	0.1324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1363	1.7571	0.0141	0.0366	-0.0179	-5.8703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2715	4.5330	0.0347	0.0949	-0.0041	4.2951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2715	4.5330	0.0347	0.0949	-0.0041	4.2951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1568	0.9677	0.0050	0.0139	-0.0005	0.1092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1393	0.9776	0.0075	0.0187	-0.0156	-5.8936
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3565	4.3579	0.0340	0.0933	-0.0012	5.6682
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3565	4.3579	0.0340	0.0933	-0.0012	5.6682
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2146	-0.3958	-0.0056	-0.0147	0.0036	0.0869
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1912	-0.3827	-0.0023	-0.0083	-0.0164	-7.9167

Worst Case Reactions LRFD

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.2429
Shear X	-0.5970
Shear Z	0.0646
Moment X	0.1773
Moment Y (Twist)	0.0310
Moment Z	13.4430

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

Result	Value (kip, kip-ft)
Axial	5.3126
Shear X	-0.3595
Shear Z	0.0413
Moment X	0.1128
Moment Y (Twist)	0.0188
Moment Z	7.9167

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	0.0067	1.7636	-0.1090	-0.2842	0.0205	-0.0337
ULS: 2. D + L	0.0067	1.7636	-0.1090	-0.2842	0.0205	-0.0337
ULS: 3. D + (S or Lr or R)	0.0102	2.5217	-0.1664	-0.4335	0.0313	-0.0623
ULS: 3. D + (S or Lr or R)	0.0067	1.7636	-0.1090	-0.2842	0.0205	-0.0337
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0093	2.3322	-0.1520	-0.3962	0.0286	-0.0551
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0067	1.7636	-0.1090	-0.2842	0.0205	-0.0337
ULS: 5b. D + 0.7E	0.0067	1.7636	-0.1090	-0.2842	0.0205	-0.0337
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0093	2.3322	-0.1520	-0.3962	0.0286	-0.0551
ULS: 8. 0.6D + 0.7E	0.0040	1.0582	-0.0654	-0.1705	0.0123	-0.0202
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2743	3.9355	-0.2781	-0.7208	0.0933	4.4412
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2743	3.9355	-0.2781	-0.7208	0.0933	4.4412
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1635	0.4715	-0.0122	-0.0337	-0.0138	0.0974
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1874	0.4752	-0.0050	-0.0172	-0.0344	-6.5696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2014	3.9611	-0.2788	-0.7237	0.0832	3.3010
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2014	3.9611	-0.2788	-0.7237	0.0832	3.3010
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1269	1.3631	-0.0794	-0.2084	0.0029	0.0432
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1448	1.3659	-0.0740	-0.1959	-0.0126	-4.9571
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2041	3.3925	-0.2358	-0.6116	0.0751	3.3225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2041	3.3925	-0.2358	-0.6116	0.0751	3.3225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1243	0.7945	-0.0364	-0.0964	-0.0052	0.0646
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1422	0.7973	-0.0310	-0.0839	-0.0207	-4.9356
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2770	3.2301	-0.2344	-0.6071	0.0851	4.4546
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2770	3.2301	-0.2344	-0.6071	0.0851	4.4546
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1608	-0.2340	0.0314	0.0799	-0.0220	0.1109
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1847	-0.2302	0.0386	0.0965	-0.0426	-6.5562

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	6.1150
Shear X	-0.4683
Shear Z	-0.4425
Moment X	-1.1481
Moment Y (Twist)	0.1523
Moment Z	11.1416

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	3.9611
Shear X	-0.2770
Shear Z	-0.2788
Moment X	-0.7237
Moment Y (Twist)	0.0933
Moment Z	6.5696

Project Details

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



Design Input Information

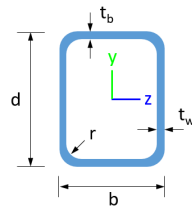
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



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



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

108	18	1.33	1.33	2.0 5	2.18,2.18,2.18,2.18,2.18,2.17,2.35,2.35,2.09,2.07,2.36,2.36,2.10,2.06,2.28,2.28,2.31,2.10,2.3 1,2.31,2.32,2.09,2.38,2.38,2.11,1.94	3 0 0	2 0 0	1
109	1	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
110	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.72,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
111	18	1.33	1.33	2.0 5	2.14,2.14,2.14,2.14,2.14,2.14,2.11,2.11,2.07,2.06,2.11,2.11,1.63,2.07,2.11,2.11,2.11,2.24,2.1 1,2.11,2.10,1.58,2.11,2.11,1.96,2.07	3 0 0	2 0 0	1
112	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
113	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.17,1.18,1.05,1.05,2.15,1.14,1.04,1.04,1.05,1.04,1.0 5,1.05,1.05,2.35,1.05,1.05,1.04,1.12	3 0 0	2 0 0	1
114	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.16,1.04,1.04,1.07,1.23,1.04,1.04,1.04,1.05,1.0 4,1.04,1.03,1.07,1.04,1.04,1.05,1.39	3 0 0	2 0 0	1
115	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.16,1.13,1.12,1.16,1.16,1.09,1.13,1.16,1.16,1.16,1.27,1.1 6,1.16,1.15,1.09,1.16,1.16,1.35,1.13	3 0 0	2 0 0	1
116	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.14,1.13,1.18,1.18,1.15,1.12,1.17,1.17,1.17,1.15,1.1 7,1.17,1.21,1.14,1.18,1.18,1.15,1.11	3 0 0	2 0 0	1
201	7	18.9 6	18.9 6	9.0 3	-	3 0 0	2 0 0	1
202	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
203	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.21,1.17,1.18,1.18,1.18,1.19,1.1 8,1.18,1.19,1.15,1.18,1.18,1.13,1.18	3 0 0	2 0 0	1
204	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.73,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
205	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.66,1.67,1.67,1.71,1.66,1.67,1.67,1.67,1.68,1.6 7,1.67,1.68,1.64,1.67,1.67,1.62,1.66	3 0 0	2 0 0	1
206	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.21,1.17,1.18,1.18,1.18,1.19,1.1 8,1.18,1.19,1.15,1.18,1.18,1.13,1.18	3 0 0	2 0 0	1
207	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.66,1.67,1.67,1.71,1.66,1.67,1.67,1.67,1.68,1.6 7,1.67,1.68,1.64,1.67,1.67,1.62,1.66	3 0 0	2 0 0	1
208	18	1.33	1.33	2.0 5	2.08,2.08,2.08,2.08,2.08,2.08,2.07,2.07,2.10,2.10,2.07,2.07,2.09,2.10,2.07,2.07,2.07,2.08,2.0 7,2.07,2.00,2.09,2.07,2.07,2.09,2.11	3 0 0	2 0 0	1
209	1	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
210	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.73,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
211	18	1.33	1.33	2.0 5	2.08,2.08,2.08,2.08,2.08,2.08,2.08,2.08,2.11,2.10,2.08,2.08,2.36,2.09,2.08,2.08,2.08,2.06,2.0 8,2.08,2.09,2.12,2.08,2.08,1.77,2.09	3 0 0	2 0 0	1
212	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
213	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.10,1.10,1.04,1.04,1.33,1.09,1.04,1.04,1.04,1.03,1.0 4,1.04,1.04,1.34,1.04,1.04,1.03,1.07	3 0 0	2 0 0	1
214	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.10,1.10,1.04,1.04,1.06,1.13,1.04,1.04,1.04,1.04,1.0 4,1.04,1.03,1.04,1.04,1.04,1.04,1.19	3 0 0	2 0 0	1
215	18	6.63	6.63	10. 20	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.17,1.16,1.15,1.15,1.19,1.16,1.15,1.15,1.16,1.14,1.1 5,1.15,1.16,1.18,1.15,1.15,1.14,1.16	3 0 0	2 0 0	1
216	18	6.63	6.63	10. 20	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.17,1.16,1.15,1.15,1.16,1.17,1.15,1.15,1.15,1.15,1.1 5,1.15,1.13,1.16,1.15,1.15,1.16,1.17	3 0 0	2 0 0	1
301	7	18.9 6	18.9 6	9.0 3	-	3 0 0	2 0 0	1

302	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
303	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.22,1.17,1.18,1.18,1.18,1.19,1.1 8,1.18,1.19,1.16,1.18,1.18,1.13,1.18	3 0 0	2 0 0	1
304	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.72,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
305	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.66,1.67,1.67,1.72,1.66,1.67,1.67,1.67,1.68,1.6 7,1.67,1.68,1.64,1.67,1.67,1.62,1.66	3 0 0	2 0 0	1
306	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.22,1.17,1.18,1.18,1.18,1.19,1.1 8,1.18,1.19,1.14,1.18,1.18,1.15,1.17	3 0 0	2 0 0	1
307	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.66,1.67,1.67,1.72,1.66,1.67,1.67,1.67,1.68,1.6 7,1.67,1.68,1.63,1.67,1.67,1.63,1.66	3 0 0	2 0 0	1
308	18	1.33	1.33	2.0 5	2.09,2.09,2.09,2.09,2.09,2.09,2.11,2.11,1.88,1.70,2.11,2.11,2.03,1.62,2.10,2.10,2.11,2.06,2.1 0,2.10,2.35,1.89,2.11,2.11,2.07,1.50	3 0 0	2 0 0	1
309	1	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
310	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.72,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
311	18	1.33	1.33	2.0 5	2.09,2.09,2.09,2.09,2.09,2.09,2.07,2.07,1.72,1.64,2.07,2.07,1.34,1.71,2.08,2.08,2.08,2.23,2.0 8,2.08,2.07,1.29,2.07,2.07,1.89,1.77	3 0 0	2 0 0	1
312	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
313	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.17,1.18,1.05,1.05,2.15,1.14,1.04,1.04,1.05,1.04,1.0 5,1.05,1.05,2.36,1.05,1.05,1.04,1.12	3 0 0	2 0 0	1
314	18	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.16,1.04,1.04,1.07,1.23,1.04,1.04,1.04,1.05,1.0 4,1.04,1.03,1.07,1.04,1.04,1.05,1.39	3 0 0	2 0 0	1
315	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.14,1.17,1.17,1.11,1.15,1.17,1.17,1.17,1.22,1.1 7,1.17,1.16,1.11,1.17,1.17,1.27,1.15	3 0 0	2 0 0	1
316	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.18,1.16,1.15,1.18,1.18,1.16,1.14,1.18,1.18,1.18,1.16,1.1 8,1.18,1.20,1.16,1.18,1.18,1.17,1.13	3 0 0	2 0 0	1
401	7	18.9 6	18.9 6	9.0 3	-	3 0 0	2 0 0	1
402	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
403	15	0.92	0.92	1.4 2	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.20,1.18,1.19,1.19,1.23,1.18,1.19,1.19,1.19,1.19,1.1 9,1.19,1.19,1.16,1.18,1.18,1.13,1.18	3 0 0	2 0 0	1
404	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.68,1.67,1.6 7,1.67,1.73,1.68,1.67,1.67,1.66,1.68	3 0 0	2 0 0	1
405	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.66,1.67,1.67,1.72,1.66,1.67,1.67,1.67,1.68,1.6 7,1.67,1.68,1.64,1.67,1.67,1.61,1.66	3 0 0	2 0 0	1
406	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.19,1.16,1.17,1.17,1.21,1.16,1.17,1.17,1.18,1.20,1.1 7,1.17,1.19,1.13,1.17,1.17,1.03,1.17	3 0 0	2 0 0	1
407	15	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.73,1.66,1.67,1.67,1.67,1.69,1.6 7,1.67,1.68,1.62,1.67,1.67,1.56,1.66	3 0 0	2 0 0	1
408	18	4.20	4.20	2.0 0	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 3,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1
409	1	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
410	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.69,1.68,1.6 8,1.68,1.77,1.68,1.67,1.67,1.65,1.68	3 0 0	2 0 0	1
411	18	4.20	4.20	2.0 0	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 3,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1
						3	2	7

412	4	1.30	1.30	2.00	-	0	0	1
413	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.17,1.33,1.35,1.17,1.17,1.77,1.31,1.17,1.17,1.17,1.12,1.17,1.17,1.19,1.80,1.17,1.17,1.43,1.26	300	200	1
414	18	4.88	4.00	7.50	1.15,1.15,1.15,1.16,1.15,1.15,1.14,1.14,1.31,1.32,1.14,1.14,1.24,1.40,1.15,1.15,1.13,1.19,1.14,1.14,1.12,1.22,1.14,1.14,1.21,1.53	300	200	1
415	18	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.11,1.11,1.09,1.09,1.16,1.11,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.17,1.09,1.09,1.11,1.10	300	200	1
416	18	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.11,1.09,1.09,1.09,1.12,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.10,1.09,1.09,1.13	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	118.57	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.72	6.45	30.09	45.74
14	120.60	98.23	19.72	6.45	30.09	45.74
15	120.60	96.18	23.36	6.45	30.09	45.74
16	120.60	96.18	23.36	6.45	30.09	45.74
101	251.16	118.57	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.31	6.45	30.09	45.74
114	120.60	98.23	18.13	6.45	30.09	45.74
115	120.60	68.63	14.89	6.45	30.09	45.74
116	120.60	68.63	15.16	6.45	30.09	45.74
201	251.16	118.57	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61

208	120.60	117.88	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	117.88	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	18.13	6.45	30.09	45.74
214	120.60	98.23	18.13	6.45	30.09	45.74
215	120.60	68.63	15.57	6.45	30.09	45.74
216	120.60	68.63	15.44	6.45	30.09	45.74
301	251.16	118.57	42.30	42.30	75.35	75.35
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	4.60	29.14	16.61
304	79.65	72.01	10.99	4.60	29.14	16.61
305	79.65	73.44	10.99	4.60	29.14	16.61
306	79.65	74.02	10.99	4.60	29.14	16.61
307	79.65	73.44	10.99	4.60	29.14	16.61
308	120.60	117.88	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	4.60	29.14	16.61
311	120.60	117.88	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	98.23	18.31	6.45	30.09	45.74
314	120.60	98.23	18.13	6.45	30.09	45.74
315	120.60	68.63	15.16	6.45	30.09	45.74
316	120.60	68.63	15.44	6.45	30.09	45.74
401	251.16	118.57	42.30	42.30	75.35	75.35
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.02	10.99	4.60	29.14	16.61
404	79.65	72.01	10.99	4.60	29.14	16.61
405	79.65	73.44	10.99	4.60	29.14	16.61
406	79.65	74.02	10.99	4.60	29.14	16.61
407	79.65	73.44	10.99	4.60	29.14	16.61
408	120.60	96.18	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.01	10.99	4.60	29.14	16.61
411	120.60	96.18	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	98.23	19.72	6.45	30.09	45.74
414	120.60	98.23	19.72	6.45	30.09	45.74
415	120.60	68.63	14.89	6.45	30.09	45.74
416	120.60	68.63	14.89	6.45	30.09	45.74

Design Ratio

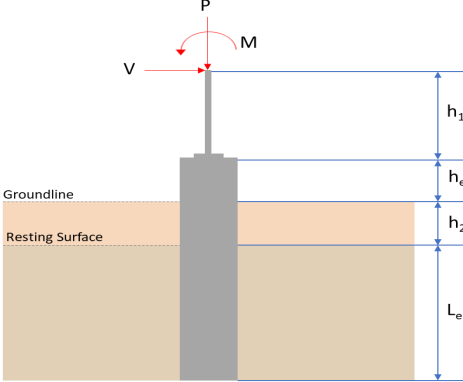
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.052	0.263	0.067	0.006	0.006	0.267	#32	0.507	Not Required	Pass
2	0.001	0.234	0.024	0.056	0.004	0.260	#13	0.052	Not Required	Pass
3	0.002	0.468	0.031	0.046	0.004	0.500	#13	0.044	Not Required	Pass
4	0.001	0.347	0.063	0.035	0.008	0.411	#13	0.117	Not Required	Pass
5	0.002	0.289	0.021	0.047	0.005	0.311	#13	0.073	Not Required	Pass
6	0.002	0.665	0.055	0.069	0.009	0.720	#13	0.044	Not Required	Pass
7	0.002	0.412	0.057	0.067	0.009	0.436	#13	0.073	Not Required	Pass
8	0.002	0.094	0.027	0.029	0.002	0.110	#13	0.088	Not Required	Pass

9	0.001	0.081	0.029	0.004	0.002	0.110	#13	0.198	Not Required	Pass
10	0.002	0.502	0.051	0.051	0.006	0.550	#13	0.078	Not Required	Pass
11	0.002	0.115	0.027	0.040	0.002	0.126	#13	0.088	Not Required	Pass
12	0.002	0.410	0.034	0.082	0.007	0.445	#13	0.052	Not Required	Pass
13	0.002	0.102	0.048	0.053	0.002	0.125	#13	0.265	Not Required	Pass
14	0.002	0.075	0.048	0.039	0.002	0.101	#13	0.177	Not Required	Pass
15	0.000	0.019	0.004	0.014	0.001	0.021	#13	Not Required	Not Required	Pass
16	0.000	0.014	0.004	0.011	0.001	0.017	#13	Not Required	Not Required	Pass
101	0.070	0.318	0.010	0.008	0.001	0.319	#16	0.507	Not Required	Pass
102	0.000	0.461	0.041	0.096	0.007	0.502	#13	0.052	Not Required	Pass
103	0.002	0.775	0.023	0.079	0.003	0.797	#13	0.044	Not Required	Pass
104	0.002	0.606	0.040	0.061	0.005	0.625	#13	0.078	Not Required	Pass
105	0.002	0.480	0.049	0.078	0.007	0.493	#13	0.073	Not Required	Pass
106	0.002	0.754	0.019	0.076	0.004	0.755	#13	0.044	Not Required	Pass
107	0.002	0.468	0.036	0.076	0.005	0.473	#13	0.073	Not Required	Pass
108	0.002	0.041	0.015	0.035	0.002	0.052	#13	0.088	Not Required	Pass
109	0.002	0.080	0.016	0.001	0.001	0.097	#13	0.198	Not Required	Pass
110	0.002	0.576	0.042	0.058	0.005	0.601	#13	0.078	Not Required	Pass
111	0.001	0.052	0.015	0.045	0.002	0.057	#13	0.088	Not Required	Pass
112	0.001	0.436	0.040	0.092	0.007	0.477	#13	0.052	Not Required	Pass
113	0.002	0.219	0.045	0.061	0.002	0.243	#13	0.265	Not Required	Pass
114	0.003	0.187	0.046	0.048	0.002	0.215	#13	0.265	Not Required	Pass
115	0.003	0.252	0.022	0.048	0.002	0.270	#13	0.439	Not Required	Pass
116	0.003	0.193	0.023	0.038	0.002	0.211	#13	0.439	Not Required	Pass
201	0.066	0.326	0.000	0.008	0.000	0.327	#16	0.507	Not Required	Pass
202	0.000	0.426	0.039	0.089	0.007	0.465	#13	0.034	Not Required	Pass
203	0.002	0.735	0.017	0.075	0.002	0.748	#13	0.044	Not Required	Pass
204	0.002	0.555	0.035	0.056	0.004	0.573	#13	0.078	Not Required	Pass
205	0.002	0.456	0.037	0.074	0.005	0.463	#13	0.073	Not Required	Pass
206	0.002	0.735	0.017	0.075	0.002	0.748	#13	0.044	Not Required	Pass
207	0.002	0.456	0.037	0.074	0.005	0.463	#13	0.073	Not Required	Pass
208	0.002	0.045	0.013	0.033	0.002	0.050	#13	0.088	Not Required	Pass
209	0.001	0.073	0.010	0.001	0.000	0.083	#13	0.198	Not Required	Pass
210	0.002	0.555	0.035	0.056	0.004	0.573	#13	0.078	Not Required	Pass
211	0.001	0.057	0.014	0.044	0.002	0.062	#13	0.088	Not Required	Pass
212	0.000	0.426	0.039	0.089	0.007	0.465	#13	0.034	Not Required	Pass
213	0.002	0.204	0.036	0.057	0.002	0.221	#13	0.265	Not Required	Pass
214	0.003	0.161	0.036	0.043	0.002	0.171	#13	0.265	Not Required	Pass
215	0.002	0.196	0.021	0.044	0.002	0.212	#13	0.439	Not Required	Pass
216	0.003	0.146	0.021	0.033	0.002	0.163	#13	0.439	Not Required	Pass
301	0.070	0.318	0.010	0.008	0.001	0.319	#16	0.507	Not Required	Pass
302	0.001	0.436	0.040	0.092	0.007	0.477	#13	0.052	Not Required	Pass
303	0.002	0.754	0.019	0.076	0.004	0.755	#13	0.044	Not Required	Pass
304	0.002	0.576	0.042	0.058	0.005	0.601	#13	0.078	Not Required	Pass
305	0.002	0.468	0.036	0.076	0.005	0.473	#13	0.073	Not Required	Pass
306	0.002	0.775	0.023	0.079	0.003	0.797	#13	0.044	Not Required	Pass
307	0.002	0.480	0.049	0.078	0.007	0.493	#13	0.073	Not Required	Pass
308	0.002	0.049	0.023	0.038	0.002	0.061	#13	0.088	Not Required	Pass
309	0.002	0.080	0.016	0.001	0.001	0.097	#13	0.198	Not Required	Pass
310	0.002	0.606	0.040	0.061	0.005	0.625	#13	0.078	Not Required	Pass
311	0.002	0.073	0.022	0.048	0.002	0.078	#13	0.088	Not Required	Pass
312	0.000	0.461	0.041	0.096	0.007	0.502	#13	0.052	Not Required	Pass
313	0.002	0.219	0.045	0.061	0.002	0.243	#13	0.265	Not Required	Pass
314	0.003	0.187	0.046	0.048	0.002	0.215	#13	0.265	Not Required	Pass

315	0.002	0.193	0.021	0.045	0.002	0.209	#13	0.439	Not Required	Pass
316	0.003	0.145	0.021	0.035	0.002	0.160	#13	0.439	Not Required	Pass
401	0.052	0.263	0.067	0.006	0.006	0.267	#32	0.507	Not Required	Pass
402	0.002	0.410	0.034	0.082	0.007	0.445	#13	0.052	Not Required	Pass
403	0.002	0.665	0.055	0.069	0.009	0.720	#13	0.044	Not Required	Pass
404	0.002	0.502	0.051	0.051	0.006	0.550	#13	0.078	Not Required	Pass
405	0.002	0.412	0.057	0.067	0.009	0.436	#13	0.073	Not Required	Pass
406	0.002	0.468	0.031	0.046	0.004	0.500	#13	0.044	Not Required	Pass
407	0.002	0.289	0.021	0.047	0.005	0.311	#13	0.073	Not Required	Pass
408	0.000	0.014	0.004	0.011	0.001	0.017	#13	Not Required	Not Required	Pass
409	0.001	0.081	0.029	0.004	0.002	0.110	#13	0.198	Not Required	Pass
410	0.001	0.347	0.063	0.035	0.008	0.411	#13	0.117	Not Required	Pass
411	0.000	0.019	0.004	0.014	0.001	0.021	#13	Not Required	Not Required	Pass
412	0.001	0.234	0.024	0.056	0.004	0.260	#13	0.052	Not Required	Pass
413	0.002	0.102	0.048	0.053	0.002	0.125	#13	0.177	Not Required	Pass
414	0.002	0.075	0.048	0.039	0.002	0.101	#13	0.265	Not Required	Pass
415	0.003	0.270	0.027	0.040	0.002	0.286	#13	0.439	Not Required	Pass
416	0.003	0.208	0.027	0.029	0.002	0.228	#13	0.439	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>3.961</td> <td>6.115</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.277</td> <td>-0.468</td> </tr> <tr> <td>V_z (kip)</td> <td>0.279</td> <td>0.443</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.724</td> <td>1.148</td> </tr> <tr> <td>M_z (kipft)</td> <td>6.570</td> <td>11.142</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	3.961	6.115	V_x (kip)	-0.277	-0.468	V_z (kip)	0.279	0.443	M_x (kipft)	0.724	1.148	M_z (kipft)	6.570	11.142	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
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V_z (kip)	0.279	0.443																										
M_x (kipft)	0.724	1.148																										
M_z (kipft)	6.570	11.142																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.277 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.044108 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(6.57 \text{ kipft}) + ((-0.277 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0462 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.1727 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.279 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.044427 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.724 \text{ kipft}) + ((0.279 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11529 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.5162 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.1727 \text{ ft}), (2.5162 \text{ ft})]$$

$$L_{e,req} = 4.173 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.173 \text{ ft})}{(4.25 \text{ ft})}$$

$$\text{Ratio} = 0.98188$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(3.961 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.24756 \text{ kip/ft}^2$$

$$q = 0.24756 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.24756 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12378$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.044108 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.0462 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.0462 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.0462 \text{ kipft/ft})) + (4 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8711 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.0462 \text{ kipft/ft})) + (3 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 [(3 \times (1.0462 \text{ kipft/ft})) + (2 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.19714 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.0462 \text{ kipft/ft})) + ((-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.63277 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.8711 \text{ ft})}{2}$$

$$p_a = 0.21533 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19714 \text{ kip/ft}^2)}{(0.21533 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91552$$

p_a - Allowable lateral soil pressure at depth L_e ,

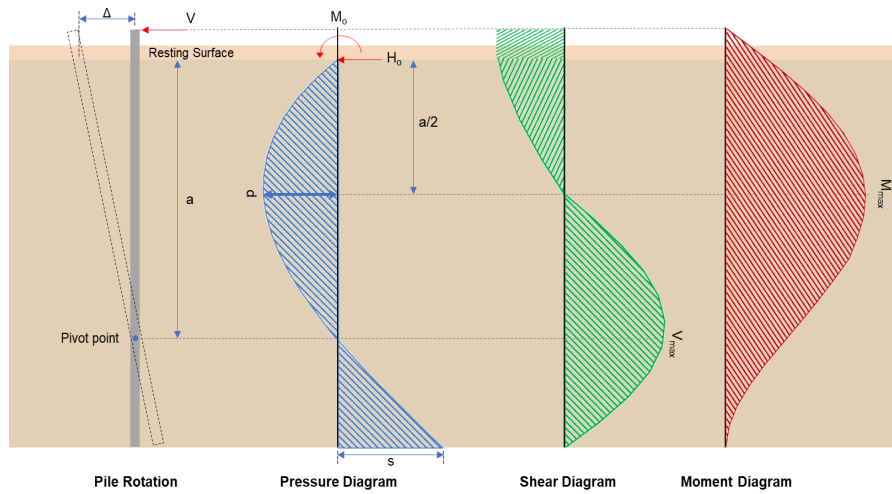
Status: **PASS**
Ratio: **0.920**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$ $p_s = 0.6375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.63277 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99258$	Status: PASS Ratio: 0.990
	<p>Considering z-direction:</p> <p>$H_o = 0.044427 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11529 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11529 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0.044427 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.11529 \text{ kipft/ft})) + (4 \times (0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))}$ $a = 3.0182 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.11529 \text{ kipft/ft})) + (3 \times (0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 [(3 \times (0.11529 \text{ kipft/ft})) + (2 \times (0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$ $p = 0.060602 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.11529 \text{ kipft/ft})) + ((0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$ $s = 0.13931 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.0182 \text{ ft})}{2}$ $p_a = 0.22636 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.060602 \text{ kip/ft}^2)}{(0.22636 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.26772$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$ $p_s = 0.6375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.270

$$Ratio = \frac{(0.13931 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$Ratio = 0.21853$$

Status: **PASS**
Ratio: **0.220**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.468 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.074522 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(11.142 \text{ kipft}) + ((-0.468 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7742 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.7742 \text{ kipft/ft})}{(-0.074522 \text{ kip/ft})}$$

$$E = 23.808 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7742 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.074522 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.7742 \text{ kipft/ft})) + (4 \times (-0.074522 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.871 \text{ ft}$$

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

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

$$V_{max} = ((-0.074522 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.871 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.871 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.1581 \text{ kip}$$

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

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

$$M_{max} = ((-0.074522 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(23.808 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 6.5945 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.443 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.070541 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.148 \text{ kipft}) + ((0.443 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1828 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1828 \text{ kipft/ft})}{(0.070541 \text{ kip/ft})}$$

$$E = 2.5914 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1828 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0.070541 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.1828 \text{ kipft/ft})) + (4 \times (0.070541 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 3.0183 \text{ ft}$$

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

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

$$V_{max} = ((0.070541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.0183 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.0183 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.49189 \text{ kip}$$

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

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

$$M_{max} = ((0.070541 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(2.5914 \text{ ft})}{(4.25 \text{ ft})} + \frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.93801 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.115 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.06 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.06 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.115 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0019209$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.115 \text{ kip} \rightarrow 6115 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(6115 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.61 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

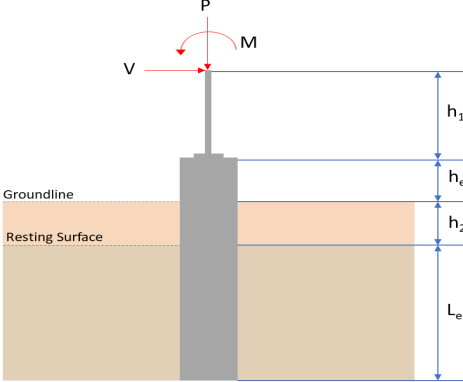
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.61 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.61 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.61 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 117.98 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.1581 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.1581 \text{ kip})}{(117.98 \text{ kip})}$ $\text{Ratio} = 0.026768$ <p>Considering z-direction:</p> <p>$V_{max} = 0.49189 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.49189 \text{ kip})}{(117.98 \text{ kip})}$ $\text{Ratio} = 0.0041694$	<p>Status: PASS Ratio: 0.030</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3ksi} \times 18432.001 in^3$ $\phi M_{n,1} = 273.423 kipft$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 ksi) \times (18432 in^3)$ $\phi M_{n,2} = 2545.9 kipft$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(273.42 kipft), (2545.9 kipft)]$ $\phi M_n = 273.42 kipft$ <p>Considering x-direction: $M_{max} = 6.5945 kipft$ - Maximum moment in the x-direction, Ratio - Capacity</p> $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(6.5945 kipft)}{(273.42 kipft)}$ $Ratio = 0.024118$	<p>Status: PASS Ratio: 0.020</p>
	<p>Considering z-direction: $M_{max} = 0.93801 kipft$ - Maximum moment in the z-direction, Ratio - Capacity</p> $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(0.93801 kipft)}{(273.42 kipft)}$ $Ratio = 0.0034306$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1192 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>3.961</td> <td>6.115</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.277</td> <td>-0.468</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.279</td> <td>-0.443</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.724</td> <td>-1.148</td> </tr> <tr> <td>M_z (kipft)</td> <td>6.570</td> <td>11.142</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	3.961	6.115	V_x (kip)	-0.277	-0.468	V_z (kip)	-0.279	-0.443	M_x (kipft)	-0.724	-1.148	M_z (kipft)	6.570	11.142	
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M_z (kipft)	6.570	11.142																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.277 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.044108 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(6.57 \text{ kipft}) + ((-0.277 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0462 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.1727 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.279 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044427 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.724 \text{ kipft}) + ((-0.279 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11529 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6803 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.1727 \text{ ft}), (1.6803 \text{ ft})]$$

$$L_{e,req} = 4.173 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.173 \text{ ft})}{(4.25 \text{ ft})}$$

$$\text{Ratio} = 0.98188$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(3.961 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.24756 \text{ kip/ft}^2$$

$$q = 0.24756 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.24756 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12378$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.044108 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.0462 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.0462 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.0462 \text{ kipft/ft})) + (4 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8711 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.0462 \text{ kipft/ft})) + (3 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 [(3 \times (1.0462 \text{ kipft/ft})) + (2 \times (-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.19714 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.0462 \text{ kipft/ft})) + ((-0.044108 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.63277 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.8711 \text{ ft})}{2}$$

$$p_a = 0.21533 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19714 \text{ kip/ft}^2)}{(0.21533 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91552$$

p_a - Allowable lateral soil pressure at depth L_e ,

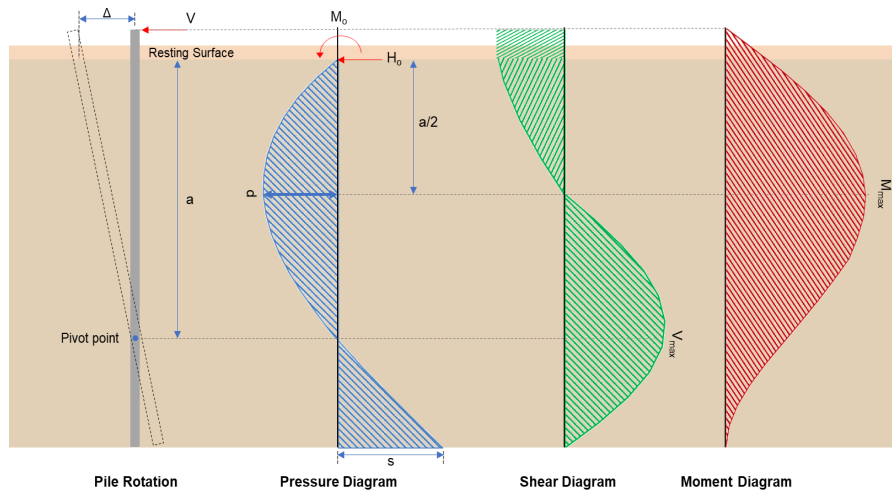
Status: **PASS**
Ratio: **0.920**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$ $p_s = 0.6375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.63277 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99258$	Status: PASS Ratio: 0.990
	<p>Considering z-direction:</p> <p>$H_o = -0.044427 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11529 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11529 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.044427 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.11529 \text{ kipft/ft})) + (4 \times (-0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))}$ $a = 3.0182 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.11529 \text{ kipft/ft})) + (3 \times (-0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 [(3 \times (0.11529 \text{ kipft/ft})) + (2 \times (-0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$ $p = -0.014491 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.11529 \text{ kipft/ft})) + ((-0.044427 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$ $s = 0.013872 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.0182 \text{ ft})}{2}$ $p_a = 0.22636 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.014491 \text{ kip/ft}^2)}{(0.22636 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.064018$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$ $p_s = 0.6375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.060

$$\text{Ratio} = \frac{(0.013872 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.02176$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.468 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.074522 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.142 \text{ kipft}) + ((-0.468 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7742 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.7742 \text{ kipft/ft})}{(-0.074522 \text{ kip/ft})}$$

$$E = 23.808 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7742 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.074522 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.7742 \text{ kipft/ft})) + (4 \times (-0.074522 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.871 \text{ ft}$$

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

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

$$V_{max} = ((-0.074522 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.871 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.871 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.1581 \text{ kip}$$

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

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

$$M_{max} = ((-0.074522 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(23.808 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.808 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.871 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 6.5945 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.443 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.070541 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.148 \text{ kipft}) + ((-0.443 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1828 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1828 \text{ kipft/ft})}{(-0.070541 \text{ kip/ft})}$$

$$E = 2.5914 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1828 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.070541 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.1828 \text{ kipft/ft})) + (4 \times (-0.070541 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 3.0183 \text{ ft}$$

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

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

$$V_{max} = ((-0.070541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.0183 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.0183 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.49189 \text{ kip}$$

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

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

$$M_{max} = ((-0.070541 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(2.5914 \text{ ft})}{(4.25 \text{ ft})} + \frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5914 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.0183 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.93801 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.115 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.06 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.06 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.115 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0019209$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.115 \text{ kip} \rightarrow 6115 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(6115 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.61 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

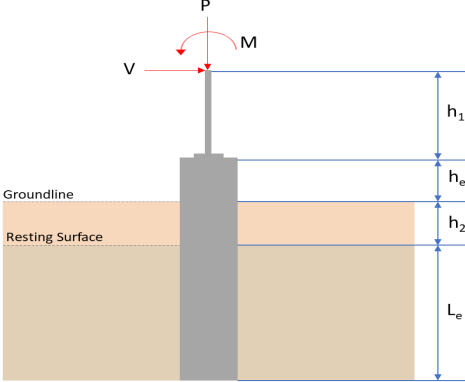
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.61 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.61 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.61 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 117.98 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.1581 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.1581 \text{ kip})}{(117.98 \text{ kip})}$ $\text{Ratio} = 0.026768$ <p>Considering z-direction:</p> <p>$V_{max} = 0.49189 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.49189 \text{ kip})}{(117.98 \text{ kip})}$ $\text{Ratio} = 0.0041694$	<p>Status: PASS Ratio: 0.030</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 6.5945\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(6.5945\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.024118$	<p>Status: PASS Ratio: 0.020</p>
	<p>Considering z-direction: $M_{max} = 0.93801\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.93801\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0034306$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.313</td> <td>8.243</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.359</td> <td>-0.597</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.041</td> <td>-0.065</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.113</td> <td>-0.177</td> </tr> <tr> <td>M_z (kipft)</td> <td>7.917</td> <td>13.443</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.313	8.243	V_x (kip)	-0.359	-0.597	V_z (kip)	-0.041	-0.065	M_x (kipft)	-0.113	-0.177	M_z (kipft)	7.917	13.443	
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M_z (kipft)	7.917	13.443																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.359 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.057166 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(7.917 \text{ kipft}) + ((-0.359 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.2607 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.4094 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.041 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0065287 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.113 \text{ kipft}) + ((-0.041 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017994 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0138 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.4094 \text{ ft}), (1.0138 \text{ ft})]$$

$$L_{e,req} = 4.409 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.409 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.97978$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.313 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33206 \text{ kip/ft}^2$$

$$q = 0.33206 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33206 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16603$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.057166 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.2607 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.2607 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.2607 \text{ kipft/ft})) + (4 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0449 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.2607 \text{ kipft/ft})) + (3 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.2607 \text{ kipft/ft})) + (2 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.20676 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.2607 \text{ kipft/ft})) + ((-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.67084 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0449 \text{ ft})}{2}$$

$$p_a = 0.22837 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20676 \text{ kip/ft}^2)}{(0.22837 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90538$$

p_a - Allowable lateral soil pressure at depth L_e ,

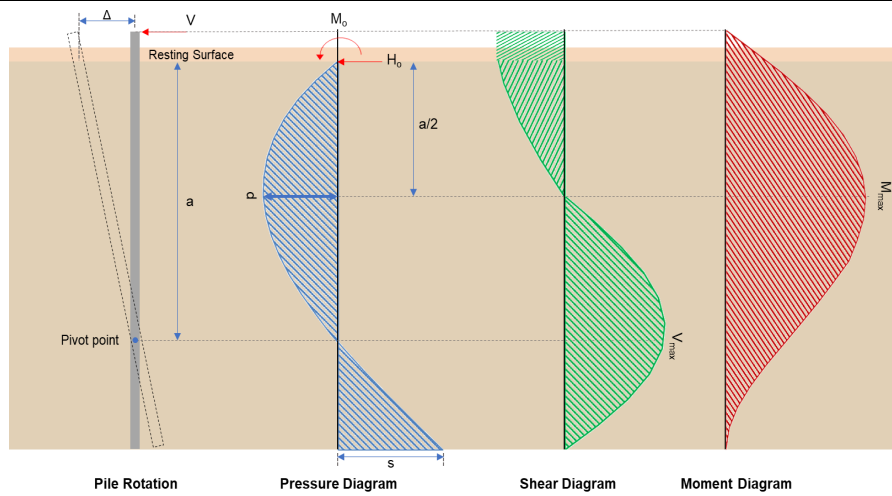
Status: **PASS**
Ratio: **0.910**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 0.675 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.67084 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99384$	<p>Status: PASS Ratio: 0.990</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.0065287 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.017994 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.017994 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.017994 \text{ kipft/ft})) + (4 \times (-0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))}$ $a = 3.1954 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.017994 \text{ kipft/ft})) + (3 \times (-0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.017994 \text{ kipft/ft})) + (2 \times (-0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$ $p = -0.0020253 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.017994 \text{ kipft/ft})) + ((-0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$ $s = 0.001958 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.1954 \text{ ft})}{2}$ $p_a = 0.23966 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0020253 \text{ kip/ft}^2)}{(0.23966 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0084507$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 0.675 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: -0.010</p>

$$Ratio = \frac{(0.001958 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.0029008$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.597 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.095064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.443 \text{ kipft}) + ((-0.597 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1406 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1406 \text{ kipft/ft})}{(-0.095064 \text{ kip/ft})}$$

$$E = 22.518 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1406 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.095064 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (2.1406 \text{ kipft/ft})) + (4 \times (-0.095064 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0441 \text{ ft}$$

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

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

$$V_{max} = ((-0.095064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0441 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0441 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.6246 \text{ kip}$$

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

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

$$M_{max} = ((-0.095064 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(22.518 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 7.9983 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.065 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01035 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.177 \text{ kipft}) + ((-0.065 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.028185 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.028185 \text{ kipft/ft})}{(-0.01035 \text{ kip/ft})}$$

$$E = 2.7231 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.028185 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.01035 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.028185 \text{ kipft/ft})) + (4 \times (-0.01035 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1966 \text{ ft}$$

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

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

$$V_{max} = ((-0.01035 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1966 \text{ ft})}{(4.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1966 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.071838 \text{ kip}$$

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

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

$$M_{max} = ((-0.01035 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(2.7231 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.14497 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.243 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.99 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.99 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.243 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0025894$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 8.243 \text{ kip} \rightarrow 8243 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(8243 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.89 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

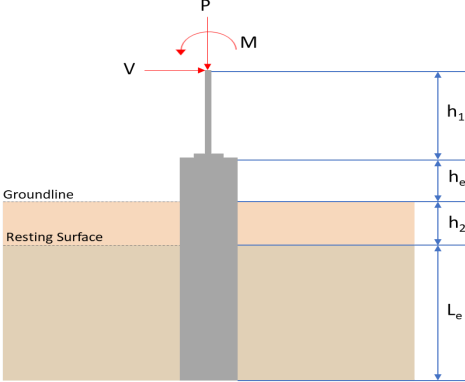
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.89 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.89 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.89 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.16 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.6246 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.6246 \text{ kip})}{(118.16 \text{ kip})}$ $\text{Ratio} = 0.030675$ <p>Considering z-direction:</p> <p>$V_{max} = 0.071838 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.071838 \text{ kip})}{(118.16 \text{ kip})}$ $\text{Ratio} = 0.00060797$	<p>Status: PASS Ratio: 0.030</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 7.9983\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(7.9983\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.029252$	<p>Status: PASS Ratio: 0.030</p>
	<p>Considering z-direction: $M_{max} = 0.14497\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.14497\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00053022$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.313</td> <td>8.243</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.359</td> <td>-0.597</td> </tr> <tr> <td>V_z (kip)</td> <td>0.041</td> <td>0.065</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.113</td> <td>0.177</td> </tr> <tr> <td>M_z (kipft)</td> <td>7.917</td> <td>13.443</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.313	8.243	V_x (kip)	-0.359	-0.597	V_z (kip)	0.041	0.065	M_x (kipft)	0.113	0.177	M_z (kipft)	7.917	13.443	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	0.113	0.177																										
M_z (kipft)	7.917	13.443																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.359 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.057166 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(7.917 \text{ kipft}) + ((-0.359 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.2607 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.4094 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.041 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0065287 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.113 \text{ kipft}) + ((0.041 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017994 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2443 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.4094 \text{ ft}), (1.2443 \text{ ft})]$$

$$L_{e,req} = 4.409 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.409 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.97978$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.313 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33206 \text{ kip/ft}^2$$

$$q = 0.33206 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33206 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16603$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.057166 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.2607 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.2607 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.2607 \text{ kipft/ft})) + (4 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0449 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.2607 \text{ kipft/ft})) + (3 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.2607 \text{ kipft/ft})) + (2 \times (-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.20676 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.2607 \text{ kipft/ft})) + ((-0.057166 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.67084 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0449 \text{ ft})}{2}$$

$$p_a = 0.22837 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20676 \text{ kip/ft}^2)}{(0.22837 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90538$$

p_a - Allowable lateral soil pressure at depth L_e ,

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.67084 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99384$$

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = 0.0065287 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.017994 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017994 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.017994 \text{ kipft/ft})) + (4 \times (0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1954 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.017994 \text{ kipft/ft})) + (3 \times (0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.017994 \text{ kipft/ft})) + (2 \times (0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.0084218 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.017994 \text{ kipft/ft})) + ((0.0065287 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.019368 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.1954 \text{ ft})}{2}$$

$$p_a = 0.23966 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0084218 \text{ kip/ft}^2)}{(0.23966 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.035141$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

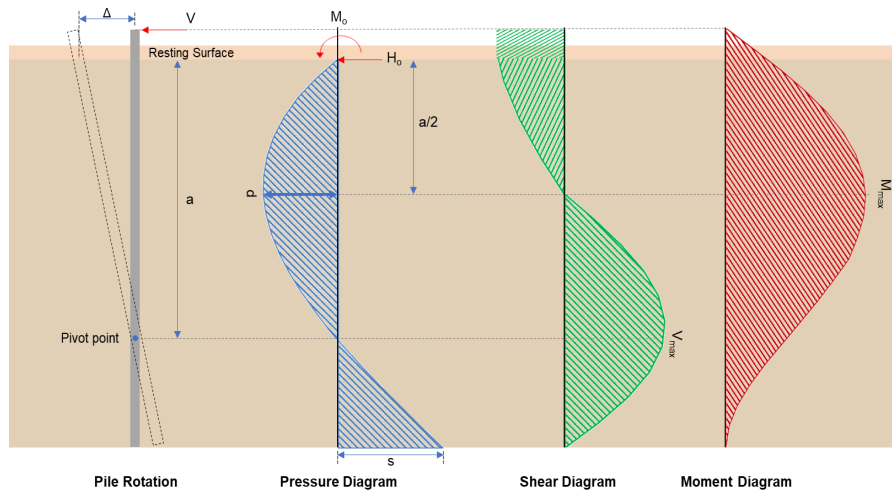
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.019368 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.028693$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.597 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.095064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.443 \text{ kipft}) + ((-0.597 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1406 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1406 \text{ kipft/ft})}{(-0.095064 \text{ kip/ft})}$$

$$E = 22.518 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1406 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.095064 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (2.1406 \text{ kipft/ft})) + (4 \times (-0.095064 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0441 \text{ ft}$$

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

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

$$V_{max} = ((-0.095064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0441 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0441 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.6246 \text{ kip}$$

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

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

$$M_{max} = ((-0.095064 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(22.518 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (22.518 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0441 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 7.9983 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.065 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.01035 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.177 \text{ kipft}) + ((0.065 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.028185 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.028185 \text{ kipft/ft})}{(0.01035 \text{ kip/ft})}$$

$$E = 2.7231 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.028185 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.01035 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.028185 \text{ kipft/ft})) + (4 \times (0.01035 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1966 \text{ ft}$$

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

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

$$V_{max} = ((0.01035 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1966 \text{ ft})}{(4.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1966 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.071838 \text{ kip}$$

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

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

$$M_{max} = ((0.01035 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(2.7231 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.7231 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1966 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.14497 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.243 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.99 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.99 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: 14 - #5 (0.625 in)

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.243 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0025894$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 8.243 \text{ kip} \rightarrow 8243 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(8243 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.89 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

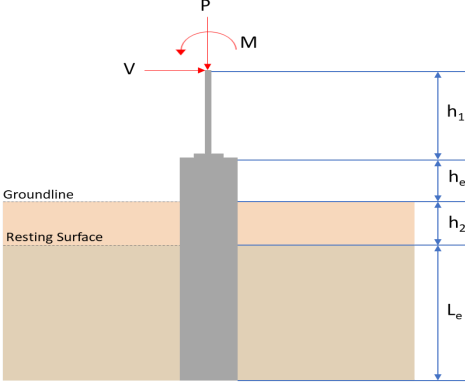
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.89 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.89 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.89 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.16 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.6246 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.6246 \text{ kip})}{(118.16 \text{ kip})}$ $\text{Ratio} = 0.030675$ <p>Considering z-direction:</p> <p>$V_{max} = 0.071838 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.071838 \text{ kip})}{(118.16 \text{ kip})}$ $\text{Ratio} = 0.00060797$	<p>Status: PASS Ratio: 0.030</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3ksi} \times 18432.001 in^3$ $\phi M_{n,1} = 273.423 kipft$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 ksi) \times (18432 in^3)$ $\phi M_{n,2} = 2545.9 kipft$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(273.42 kipft), (2545.9 kipft)]$ $\phi M_n = 273.42 kipft$ <p>Considering x-direction: $M_{max} = 7.9983 kipft$ - Maximum moment in the x-direction, Ratio - Capacity</p> $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(7.9983 kipft)}{(273.42 kipft)}$ $Ratio = 0.029252$	<p>Status: PASS Ratio: 0.030</p>
	<p>Considering z-direction: $M_{max} = 0.14497 kipft$ - Maximum moment in the z-direction, Ratio - Capacity</p> $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(0.14497 kipft)}{(273.42 kipft)}$ $Ratio = 0.00053022$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.071</td> <td>7.863</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.346</td> <td>-0.578</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.095</td> <td>13.777</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.071	7.863	V_x (kip)	-0.346	-0.578	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	8.095	13.777	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.346 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.055096 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(8.095 \text{ kipft}) + ((-0.346 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.289 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 4.4546 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(4.4546 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 4.455 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 4.75 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(4.455 \text{ ft})}{(4.75 \text{ ft})}$ $\text{Ratio} = 0.93789$	<p>Status: PASS Ratio: 0.940</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(5.071 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.31694 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.31694 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.15847$	<p>Status: PASS Ratio: 0.160</p>
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.055096$ kip/ft - Lateral force per length of pile,

$M_o = 1.289$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.289 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.055096 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.289 \text{ kipft/ft})) + (4 \times (-0.055096 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2139 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.289 \text{ kipft/ft})) + (3 \times (-0.055096 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.289 \text{ kipft/ft})) + (2 \times (-0.055096 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.18994 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.289 \text{ kipft/ft})) + ((-0.055096 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.61597 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2139 \text{ ft})}{2}$$

$$p_a = 0.24104 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18994 \text{ kip/ft}^2)}{(0.24104 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.78798$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

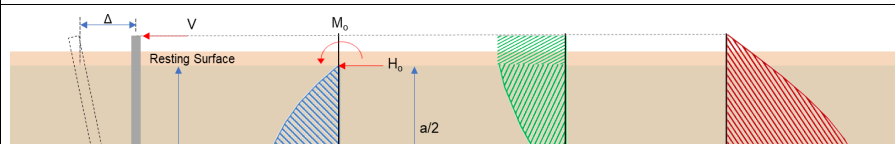
$$\text{Ratio} = \frac{s}{p_s}$$

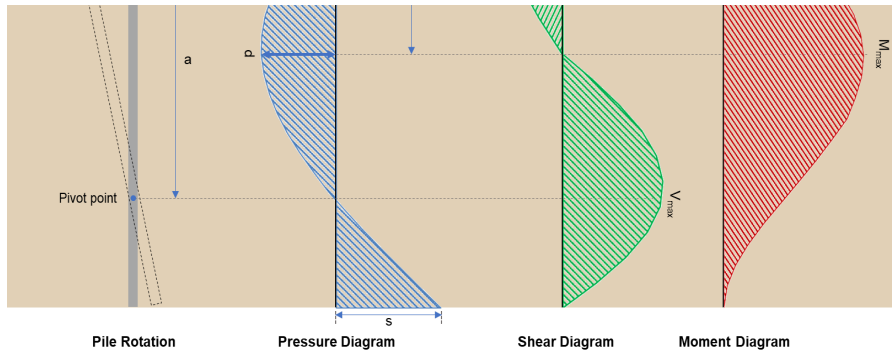
$$\text{Ratio} = \frac{(0.61597 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86453$$

Status: **PASS**
Ratio: **0.790**

Status: **PASS**
Ratio: **0.860**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.578 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.092038 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.777 \text{ kipft}) + ((-0.578 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1938 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1938 \text{ kipft/ft})}{(-0.092038 \text{ kip/ft})}$$

$$E = 23.836 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1938 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.092038 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.1938 \text{ kipft/ft})) + (4 \times (-0.092038 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2131 \text{ ft}$$

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

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

$$V_{max} = ((-0.092038 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.836 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2131 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.836 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2131 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5185 \text{ kip}$$

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

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

$$M_{max} = ((-0.092038 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(23.836 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2131 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.836 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2131 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.836 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2131 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.1959 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.863 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.863 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.00247$$

Status: **PASS**
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 7.863 \text{ kip} \rightarrow 7863 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(7863 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.84 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.84 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.84 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.84 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.13 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.5185 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.5185 \text{ kip})}{(118.13 \text{ kip})}$ $\text{Ratio} = 0.029785$	<p>Status: PASS Ratio: 0.030</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kip ft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = 0.85 f'_c S_m$	

$$\phi M_{n,z} = \phi S_x F_y$$

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

$$\phi M_{n,z} = 2545.9 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$

$$\phi M_n = 273.42 \text{ kipft}$$

Considering x-direction:

$M_{max} = 8.1959 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.1959 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.029975$$

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