

Your Project Calculations



Project Name: Sagamore Golf Club 5x10 - V1Jb

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Sagamore%20Golf%20Club%205x10%20-%20V1Jb&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=Pt4JbuPCUNGUtaVBkT00IctWcEIZGOedGLNg51iZNmRKO7sDvL3tZRVqBWO2mrug

Array Specification

Product:	Beam
Unique ID:	4P-19.75-8TOP-HD-45-L-5Hx10W-2CA7
Duty Classification:	HD
Module Width:	45.00 in
Module Length:	87.00in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	48
Front Edge Clearance:	4
Total Array Height at Tilt:	18.01 ft
Total Frame Length:	74.25 ft
Frame Weight:	3605 lbs
Array Dimensions N/S:	18.96 ft
Array Dimensions E/W:	73.33 ft
Rail Length:	227.50 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

Support Specifications

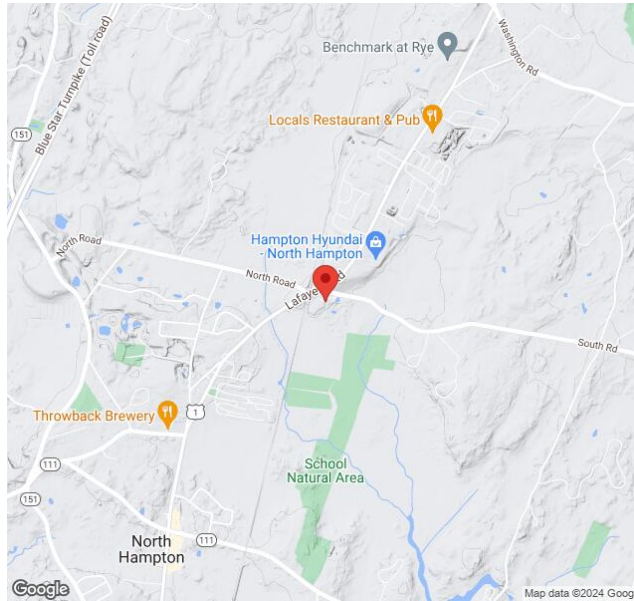
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.04 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 9.00 ft Pile 2: 9.25 ft Pile 3: 9.25 ft Pile 4: 9.00 ft
Foundation Volume:	9.556 y ³
Foundation Result:	PASSED
Mount Twist:	0.592561 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	22 North Road, North Hampton, NH 03862, USA
Wind Speed:	107 mph
Snow Load:	50 psf
Design Uplift Pressure:	0.016262 ksf
Design Downforce Pressure:	-0.016262 ksf
Design Snow Pressure:	0.012096 ksf



Design Disclaimer

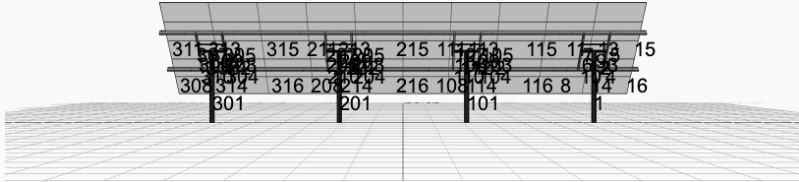
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

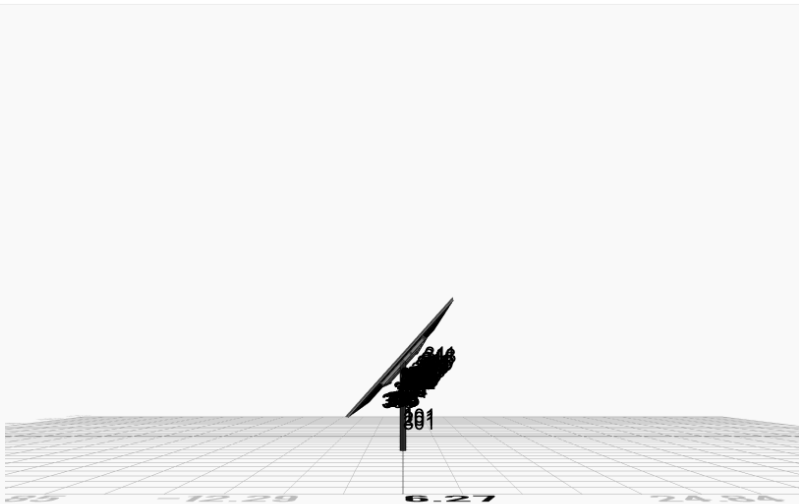
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Design Notes:

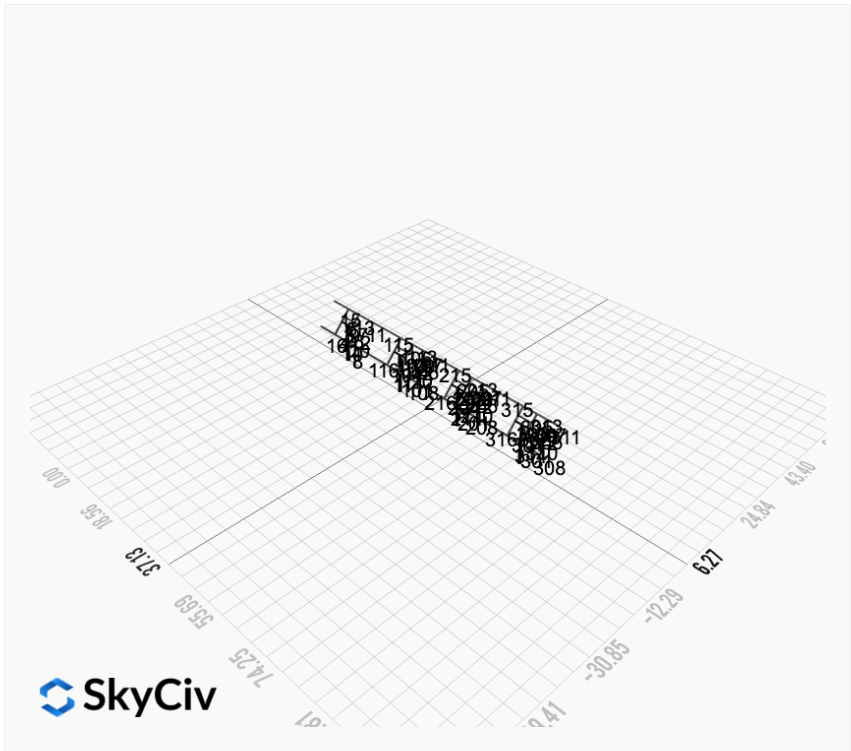
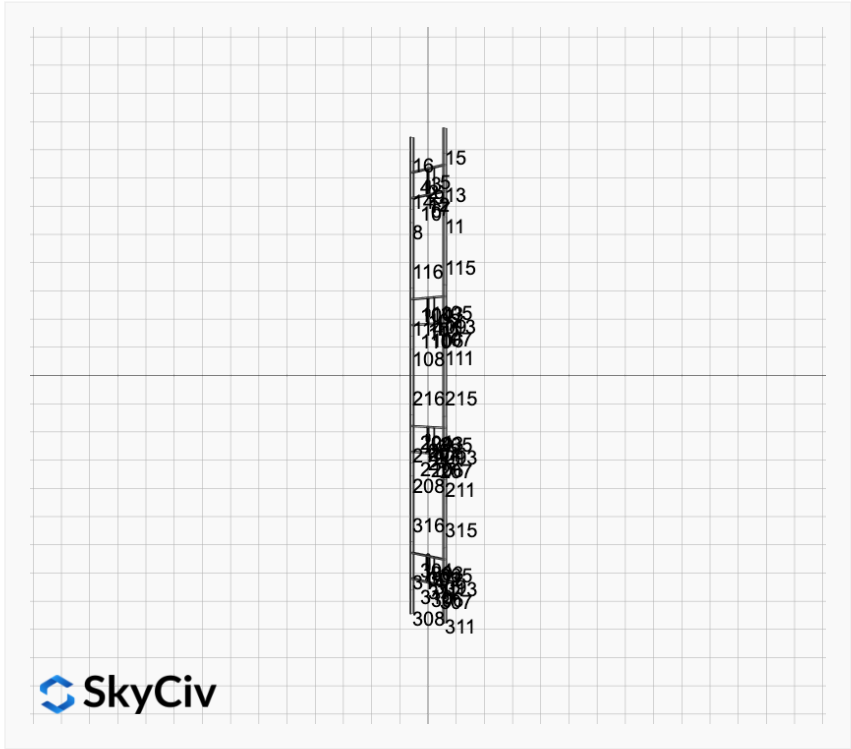
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

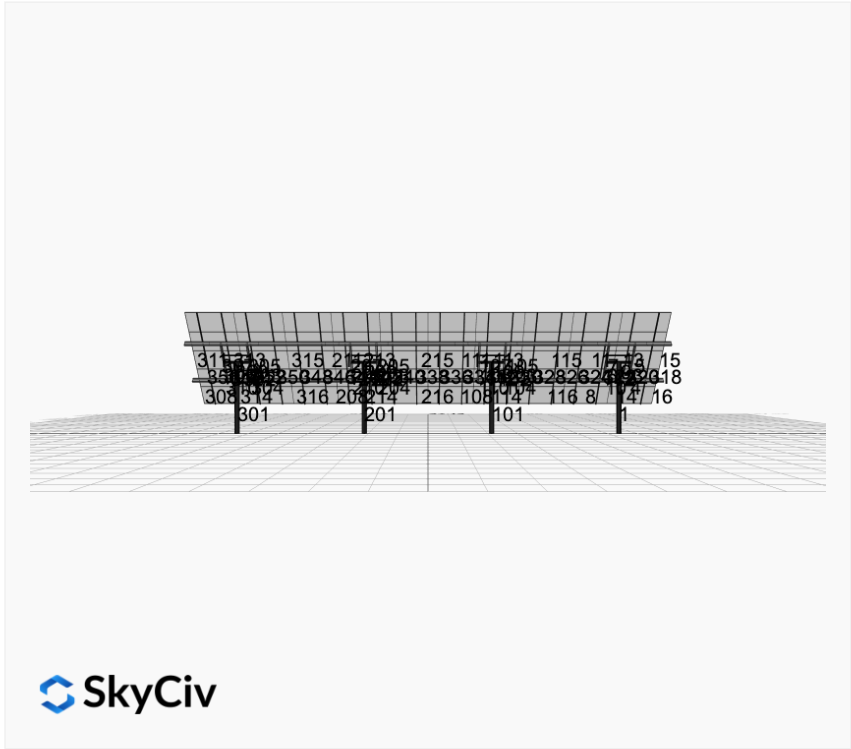


 SkyCiv

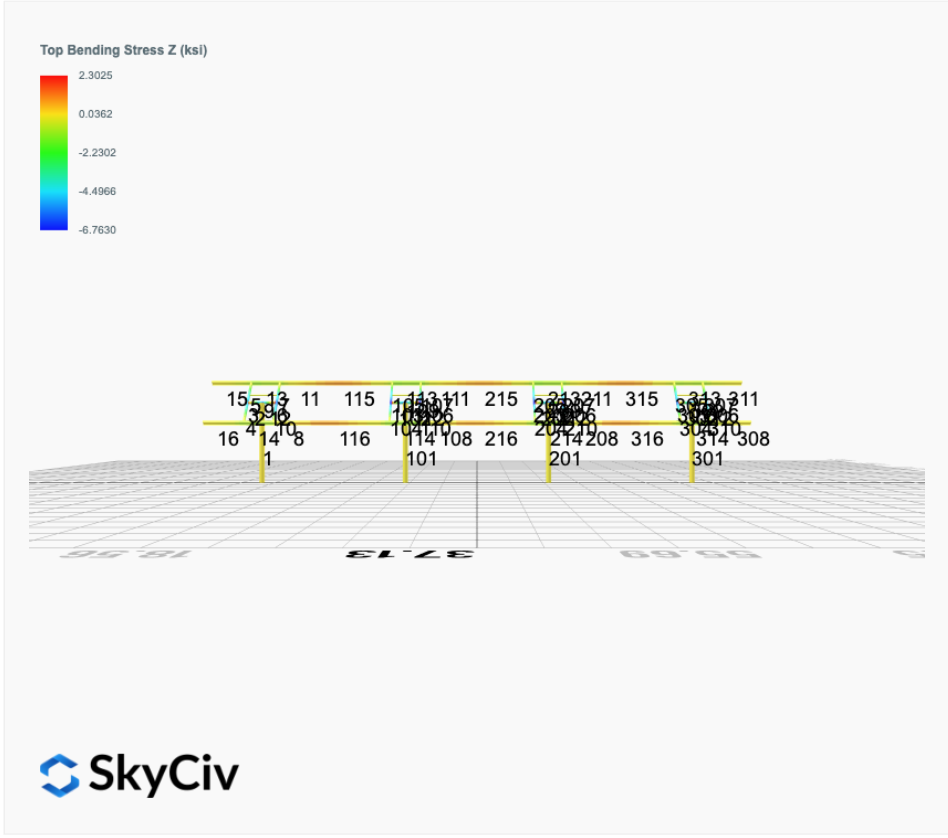
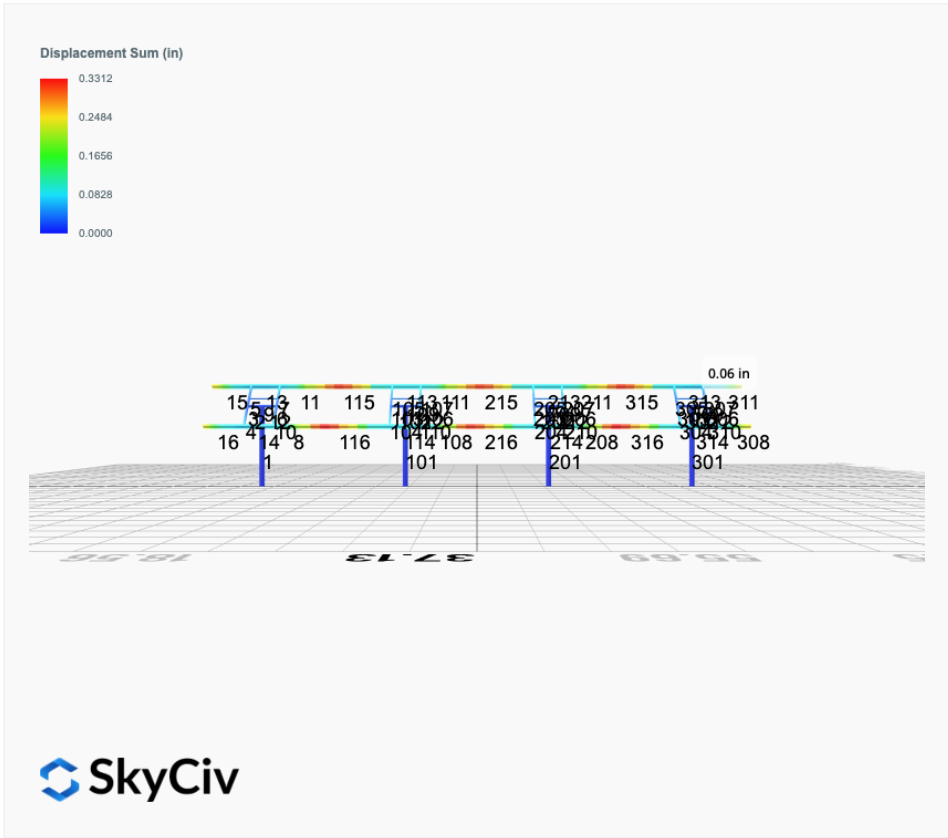


 SkyCiv

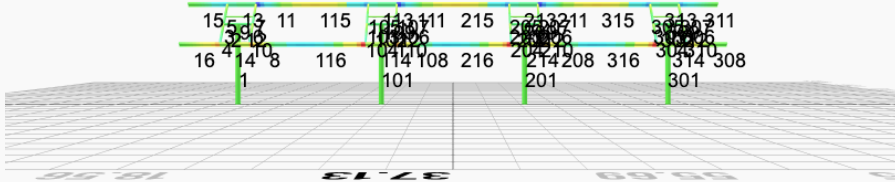




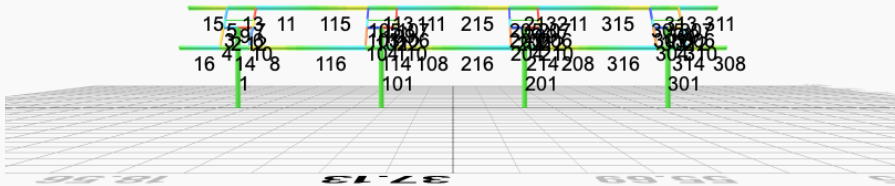
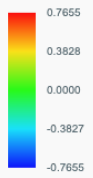
FEM Results (Envelope Worst Case for each member)



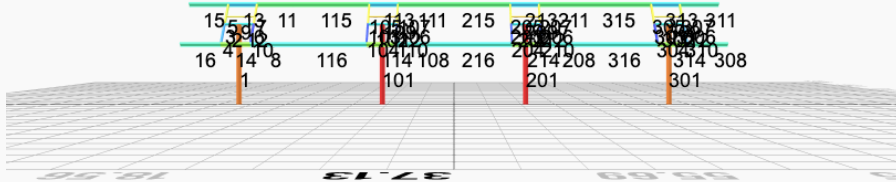
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 2. D + L	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 3. D + (S or Lr or R)	0.0220	4.9626	0.0819	0.2713	-0.0316	-0.2004
ULS: 3. D + (S or Lr or R)	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0187	4.2994	0.0696	0.2306	-0.0268	-0.1680
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 5b. D + 0.7E	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0187	4.2994	0.0696	0.2306	-0.0268	-0.1680
ULS: 8. 0.6D + 0.7E	0.0053	1.3861	0.0197	0.0652	-0.0075	-0.0424
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3524	4.4185	0.1042	0.3248	-0.3499	26.3616
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3687	0.2023	-0.0371	-0.1030	0.3184	-25.9704
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7522	5.8807	0.1231	0.3928	-0.2799	19.6562
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0187	4.2994	0.0696	0.2306	-0.0268	-0.1680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7886	2.7186	0.0172	0.0719	0.2213	-19.5928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0187	4.2994	0.0696	0.2306	-0.0268	-0.1680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7621	3.8914	0.0863	0.2708	-0.2655	19.7536
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7787	0.7292	-0.0196	-0.0501	0.2357	-19.4955
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0088	2.3101	0.0328	0.1086	-0.0124	-0.0706
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3559	3.4945	0.0910	0.2814	-0.3449	26.3899
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0053	1.3861	0.0197	0.0652	-0.0075	-0.0424
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3652	-0.7218	-0.0502	-0.1464	0.3234	-25.9421
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0053	1.3861	0.0197	0.0652	-0.0075	-0.0424

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	8.7734
Shear X	-3.9515
Shear Z	0.1841
Moment X	0.5766
Moment Y (Twist)	0.5924
Moment Z	44.4864

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	5.8807
Shear X	-2.3687
Shear Z	0.1231
Moment X	0.3928
Moment Y (Twist)	0.3499
Moment Z	26.3899

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	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 2. D + L	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 3. D + (S or Lr or R)	-0.0220	5.6174	-0.0020	-0.0068	0.0065	0.2665
ULS: 3. D + (S or Lr or R)	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0187	4.8564	-0.0017	-0.0058	0.0055	0.2288
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 5b. D + 0.7E	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0187	4.8564	-0.0017	-0.0058	0.0055	0.2288
ULS: 8. 0.6D + 0.7E	-0.0053	1.5439	-0.0005	-0.0016	0.0016	0.0693
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6880	5.0032	0.0117	0.0340	-0.0711	29.9480
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6717	0.1426	-0.0124	-0.0368	0.0719	-29.0716
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0281	6.6789	0.0077	0.0218	-0.0498	22.6031
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0187	4.8564	-0.0017	-0.0058	0.0055	0.2288
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9917	3.0334	-0.0104	-0.0314	0.0575	-21.6616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0187	4.8564	-0.0017	-0.0058	0.0055	0.2288
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0182	4.3957	0.0086	0.0248	-0.0526	22.4899
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0016	0.7503	-0.0095	-0.0283	0.0546	-21.7748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0088	2.5732	-0.0008	-0.0027	0.0026	0.1156
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6845	3.9739	0.0120	0.0351	-0.0721	29.9018
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0053	1.5439	-0.0005	-0.0016	0.0016	0.0693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6752	-0.8867	-0.0121	-0.0358	0.0709	-29.1178
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0053	1.5439	-0.0005	-0.0016	0.0016	0.0693

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.9832
Shear X	-4.4811
Shear Z	-0.0216
Moment X	-0.0642
Moment Y (Twist)	0.1245
Moment Z	50.6365

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6789
Shear X	-2.6880
Shear Z	-0.0124
Moment X	-0.0368
Moment Y (Twist)	0.0721
Moment Z	29.9480

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	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 2. D + L	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 3. D + (S or Lr or R)	-0.0220	5.6174	0.0020	0.0068	-0.0064	0.2665
ULS: 3. D + (S or Lr or R)	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0187	4.8564	0.0017	0.0058	-0.0054	0.2288
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 5b. D + 0.7E	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0187	4.8564	0.0017	0.0058	-0.0054	0.2288
ULS: 8. 0.6D + 0.7E	-0.0053	1.5439	0.0005	0.0016	-0.0015	0.0693
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6880	5.0032	-0.0117	-0.0340	0.0711	29.9480
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6717	0.1426	0.0124	0.0368	-0.0719	-29.0716
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0281	6.6789	-0.0077	-0.0218	0.0498	22.6031
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0187	4.8564	0.0017	0.0058	-0.0054	0.2288
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9917	3.0334	0.0104	0.0313	-0.0574	-21.6616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0187	4.8564	0.0017	0.0058	-0.0054	0.2288

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	-2.0182	4.3957	-0.0086	-0.0248	0.0527	22.4899
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0016	0.7503	0.0095	0.0283	-0.0546	-21.7748
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0088	2.5732	0.0008	0.0027	-0.0026	0.1156
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6845	3.9739	-0.0120	-0.0351	0.0721	29.9018
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0053	1.5439	0.0005	0.0016	-0.0015	0.0693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6752	-0.8867	0.0121	0.0358	-0.0709	-29.1178
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0053	1.5439	0.0005	0.0016	-0.0015	0.0693

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.9832
Shear X	-4.4811
Shear Z	0.0216
Moment X	0.0644
Moment Y (Twist)	0.1245
Moment Z	50.6366

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6789
Shear X	-2.6880
Shear Z	0.0124
Moment X	0.0368
Moment Y (Twist)	0.0721
Moment Z	29.9480

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	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 2. D + L	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 3. D + (S or Lr or R)	0.0220	4.9625	-0.0819	-0.2714	0.0317	-0.2004
ULS: 3. D + (S or Lr or R)	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0187	4.2994	-0.0696	-0.2307	0.0269	-0.1679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 5b. D + 0.7E	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0187	4.2994	-0.0696	-0.2307	0.0269	-0.1679
ULS: 8. 0.6D + 0.7E	0.0053	1.3861	-0.0197	-0.0652	0.0075	-0.0424
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3524	4.4185	-0.1042	-0.3249	0.3499	26.3616
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.3687	0.2023	0.0371	0.1030	-0.3184	-25.9704
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7522	5.8807	-0.1231	-0.3929	0.2800	19.6563
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0187	4.2994	-0.0696	-0.2307	0.0269	-0.1679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7886	2.7186	-0.0172	-0.0720	-0.2213	-19.5928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0187	4.2994	-0.0696	-0.2307	0.0269	-0.1679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7621	3.8914	-0.0863	-0.2708	0.2655	19.7536
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7787	0.7292	0.0196	0.0501	-0.2357	-19.4954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0088	2.3101	-0.0328	-0.1086	0.0124	-0.0706
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3559	3.4945	-0.0910	-0.2814	0.3449	26.3899
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0053	1.3861	-0.0197	-0.0652	0.0075	-0.0424
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3652	-0.7218	0.0502	0.1464	-0.3234	-25.9421
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0053	1.3861	-0.0197	-0.0652	0.0075	-0.0424

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	8.7734
Shear X	-3.9515
Shear Z	-0.1841
Moment X	-0.5769
Moment Y (Twist)	0.5926
Moment Z	44.4871

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	5.8807
Shear X	-2.3687
Shear Z	-0.1231
Moment X	-0.3929
Moment Y (Twist)	0.3499
Moment Z	26.3899

Project Details

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

User Name: sales@mtsolar.us
 Project Name: Sagamore Golf Club 5x10 - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

Section Dimensions

ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
9	8in Pipe Sch 40	8.63	0.32				

ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	

ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I _{yp} (in ⁴)	I _{zp} (in ⁴)	I _w (in ⁶)	S _{yp} (in ³)	S _{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.68	6.12	40.24	43.62
114	133.20	85.85	23.80	6.12	40.24	43.62
115	133.20	69.16	17.00	6.12	40.24	43.62
116	133.20	69.16	17.70	6.12	40.24	43.62
201	377.97	196.09	83.29	83.29	113.39	113.39
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.68	6.12	40.24	43.62
214	133.20	85.85	23.80	6.12	40.24	43.62
215	133.20	69.16	17.33	6.12	40.24	43.62
216	133.20	69.16	17.76	6.12	40.24	43.62
301	377.97	196.09	83.29	83.29	113.39	113.39
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	52.83	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	52.83	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.68	6.12	40.24	43.62
314	133.20	85.85	25.08	6.12	40.24	43.62
315	133.20	69.16	17.25	6.12	40.24	43.62
316	133.20	69.16	17.32	6.12	40.24	43.62

Design Ratio

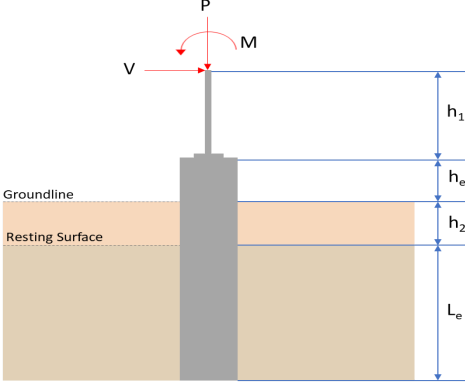
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.045	0.534	0.017	0.035	0.002	0.560	#13	0.474	Not Required	Pass
2	0.003	0.259	0.168	0.063	0.032	0.401	#13	0.035	Not Required	Pass
3	0.009	0.426	0.045	0.042	0.004	0.454	#13	0.045	Not Required	Pass

4	0.009	0.425	0.145	0.043	0.032	0.494	#21	0.080	Not Required	Pass
5	0.009	0.264	0.142	0.042	0.036	0.281	#13	0.074	Not Required	Pass
6	0.012	0.495	0.086	0.050	0.017	0.548	#13	0.045	Not Required	Pass
7	0.012	0.307	0.197	0.049	0.049	0.334	#13	0.074	Not Required	Pass
8	0.002	0.060	0.173	0.032	0.019	0.176	#23	0.095	Not Required	Pass
9	0.013	0.032	0.059	0.002	0.002	0.088	#13	0.204	Not Required	Pass
10	0.012	0.480	0.190	0.048	0.041	0.558	#21	0.080	Not Required	Pass
11	0.003	0.055	0.177	0.033	0.019	0.179	#21	0.095	Not Required	Pass
12	0.003	0.323	0.195	0.078	0.037	0.494	#13	0.035	Not Required	Pass
13	0.006	0.149	0.453	0.043	0.025	0.553	#21	0.286	Not Required	Pass
14	0.009	0.147	0.448	0.042	0.025	0.537	#21	0.190	Not Required	Pass
15	0.000	0.047	0.158	0.020	0.012	0.200	#21	Not Required	Not Required	Pass
16	0.000	0.047	0.158	0.020	0.012	0.200	#21	Not Required	Not Required	Pass
101	0.051	0.608	0.002	0.040	0.000	0.631	#13	0.474	Not Required	Pass
102	0.003	0.331	0.205	0.081	0.037	0.505	#13	0.035	Not Required	Pass
103	0.012	0.516	0.074	0.051	0.011	0.561	#13	0.045	Not Required	Pass
104	0.012	0.523	0.191	0.052	0.041	0.612	#21	0.080	Not Required	Pass
105	0.012	0.320	0.198	0.051	0.050	0.348	#13	0.074	Not Required	Pass
106	0.012	0.529	0.074	0.053	0.011	0.572	#13	0.045	Not Required	Pass
107	0.012	0.328	0.194	0.052	0.049	0.356	#13	0.074	Not Required	Pass
108	0.002	0.043	0.169	0.033	0.019	0.195	#21	0.095	Not Required	Pass
109	0.015	0.035	0.045	0.001	0.000	0.084	#13	0.204	Not Required	Pass
110	0.012	0.527	0.187	0.053	0.040	0.609	#21	0.080	Not Required	Pass
111	0.003	0.050	0.173	0.033	0.019	0.195	#21	0.095	Not Required	Pass
112	0.003	0.334	0.211	0.081	0.038	0.518	#13	0.035	Not Required	Pass
113	0.006	0.152	0.457	0.043	0.025	0.584	#21	0.286	Not Required	Pass
114	0.009	0.169	0.453	0.044	0.025	0.589	#21	0.286	Not Required	Pass
115	0.005	0.205	0.249	0.033	0.019	0.438	#21	0.473	Not Required	Pass
116	0.002	0.202	0.250	0.034	0.019	0.434	#21	0.473	Not Required	Pass
201	0.051	0.608	0.002	0.040	0.000	0.631	#13	0.474	Not Required	Pass
202	0.003	0.334	0.211	0.081	0.038	0.518	#13	0.035	Not Required	Pass
203	0.012	0.529	0.074	0.053	0.011	0.572	#13	0.045	Not Required	Pass
204	0.012	0.527	0.187	0.053	0.040	0.609	#21	0.080	Not Required	Pass
205	0.012	0.328	0.194	0.052	0.049	0.356	#13	0.074	Not Required	Pass
206	0.012	0.516	0.074	0.051	0.011	0.561	#13	0.045	Not Required	Pass
207	0.012	0.320	0.198	0.051	0.050	0.348	#13	0.074	Not Required	Pass
208	0.002	0.049	0.177	0.034	0.019	0.199	#21	0.095	Not Required	Pass
209	0.015	0.035	0.045	0.001	0.000	0.084	#13	0.204	Not Required	Pass
210	0.012	0.523	0.191	0.052	0.041	0.612	#21	0.080	Not Required	Pass
211	0.003	0.056	0.180	0.033	0.019	0.197	#21	0.095	Not Required	Pass
212	0.003	0.331	0.205	0.081	0.037	0.505	#13	0.035	Not Required	Pass
213	0.006	0.152	0.457	0.043	0.025	0.584	#21	0.286	Not Required	Pass
214	0.009	0.169	0.453	0.044	0.025	0.589	#21	0.286	Not Required	Pass
215	0.005	0.194	0.250	0.033	0.019	0.423	#21	0.473	Not Required	Pass
216	0.002	0.179	0.250	0.033	0.019	0.412	#21	0.473	Not Required	Pass
301	0.045	0.534	0.017	0.035	0.002	0.560	#13	0.474	Not Required	Pass
302	0.003	0.323	0.195	0.078	0.037	0.494	#13	0.035	Not Required	Pass
303	0.012	0.495	0.086	0.050	0.017	0.548	#13	0.045	Not Required	Pass
304	0.012	0.480	0.190	0.048	0.041	0.558	#21	0.080	Not Required	Pass
305	0.012	0.307	0.197	0.049	0.049	0.334	#13	0.074	Not Required	Pass
306	0.009	0.426	0.045	0.042	0.004	0.454	#13	0.045	Not Required	Pass
307	0.009	0.264	0.142	0.042	0.036	0.281	#13	0.074	Not Required	Pass
308	0.000	0.047	0.158	0.020	0.012	0.200	#21	Not Required	Not Required	Pass
309	0.013	0.032	0.059	0.002	0.002	0.088	#13	0.204	Not Required	Pass

309	0.015	0.052	0.059	0.002	0.002	0.000	#13	0.204	Not Required	Pass
310	0.009	0.425	0.145	0.043	0.032	0.494	#21	0.080	Not Required	Pass
311	0.000	0.047	0.158	0.020	0.012	0.200	#21	Not Required	Not Required	Pass
312	0.003	0.259	0.168	0.063	0.032	0.401	#13	0.035	Not Required	Pass
313	0.006	0.149	0.453	0.043	0.025	0.553	#21	0.190	Not Required	Pass
314	0.009	0.147	0.448	0.042	0.025	0.537	#21	0.286	Not Required	Pass
315	0.005	0.210	0.250	0.033	0.019	0.439	#21	0.473	Not Required	Pass
316	0.002	0.207	0.250	0.032	0.019	0.437	#21	0.473	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS																										
	<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: round $D = 36$ in - Pile diameter $L = 9$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.881</td> <td>8.773</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.369</td> <td>-3.952</td> </tr> <tr> <td>V_z (kip)</td> <td>0.123</td> <td>0.184</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.393</td> <td>0.577</td> </tr> <tr> <td>M_z (kipft)</td> <td>26.390</td> <td>44.486</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.881	8.773	V_x (kip)	-2.369	-3.952	V_z (kip)	0.123	0.184	M_x (kipft)	0.393	0.577	M_z (kipft)	26.390	44.486	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	26.390	44.486																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.369 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.78967 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(26.39 \text{ kipft}) + ((-2.369 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.7967 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$$L_{e,x} = 7.9922 \text{ ft} - \text{Required depth in x-direction,}$$

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.123 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.041 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.393 \text{ kipft}) + ((0.123 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.131 \text{ kipft/ft}$$

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:

$$L_{e,z} = 3.0449 \text{ ft} - \text{Required depth in z-direction,}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9922 \text{ ft}), (3.0449 \text{ ft})]$$

$$L_{e,req} = 7.992 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.992 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.888$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(5.881 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.83199 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.83199 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.416$$

Status: **PASS**
Ratio: **0.420**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.78967 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.7967 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7967 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (8.7967 \text{ kipft/ft})) + (4 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2625 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (8.7967 \text{ kipft/ft})) + (3 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (8.7967 \text{ kipft/ft})) + (2 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.22963 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (8.7967 \text{ kipft/ft})) + ((-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2202 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2625 \text{ ft})}{2}$$

$$p_a = 0.46969 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22963 \text{ kip/ft}^2)}{(0.46969 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.48891$$

Status: **PASS**
Ratio: **0.490**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2202 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90383$$

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.041 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.131 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.131 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.041 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.131 \text{ kipft/ft})) + (4 \times (0.041 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4894 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.131 \text{ kipft/ft})) + (3 \times (0.041 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.131 \text{ kipft/ft})) + (2 \times (0.041 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.034206 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.131 \text{ kipft/ft})) + ((0.041 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 0.073422 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4894 \text{ ft})}{2}$$

$$p_a = 0.4867 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.034206 \text{ kip/ft}^2)}{(0.4867 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.070281$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

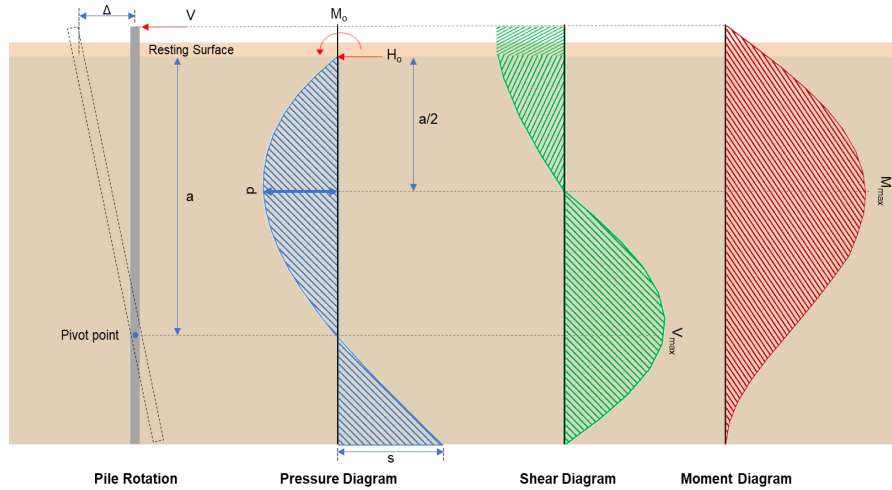
Status: **PASS**
Ratio: **0.070**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.073422 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.054387$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.952 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3173 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.486 \text{ kipft}) + ((-3.952 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.829 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.829 \text{ kipft/ft})}{(-1.3173 \text{ kip/ft})}$$

$$E = 11.257 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.829 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3173 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (14.829 \text{ kipft/ft})) + (4 \times (-1.3173 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2608 \text{ ft}$$

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

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

$$V_{max} = ((-1.3173 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2608 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2608 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.353 \text{ kip}$$

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

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

$$M_{max} = ((-1.3173 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(11.257 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 47.874 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.184 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.061333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.577 \text{ kipft}) + ((0.184 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19233 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19233 \text{ kipft/ft})}{(0.061333 \text{ kip/ft})}$$

$$E = 3.1359 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19233 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.061333 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.19233 \text{ kipft/ft})) + (4 \times (0.061333 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4926 \text{ ft}$$

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

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

$$V_{max} = ((0.061333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4926 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4926 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23672 \text{ kip}$$

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

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

$$M_{max} = ((0.061333 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.1359 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91823 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(8.773 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.099 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-37.099 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.773 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0069965$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.773 \text{ kip} \rightarrow 8773 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8773 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.927 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

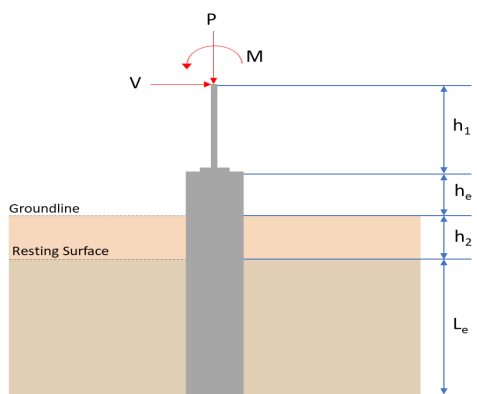
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (75.927 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 75.927 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.927 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.163 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.353 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(11.353 \text{ kip})}{(74.163 \text{ kip})}$ $Ratio = 0.15308$ <p>Considering z-direction:</p> <p>$V_{max} = 0.23672 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.23672 \text{ kip})}{(74.163 \text{ kip})}$ $Ratio = 0.0031919$	<p>Status: PASS Ratio: 0.150</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 f'_{ck} S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 47.874 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(47.874 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.77183$	<p>Status: PASS Ratio: 0.770</p>
	<p>Considering z-direction: $M_{max} = 0.91823 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.91823 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.014804$	<p>Status: PASS Ratio: 0.010</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: round $D = 36$ in - Pile diameter $L = 9$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="414 1075 1189 1176"> <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="670 1265 933 1433"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.881</td> <td>8.773</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.369</td> <td>-3.952</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.123</td> <td>-0.184</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.393</td> <td>-0.577</td> </tr> <tr> <td>M_z (kipft)</td> <td>26.390</td> <td>44.487</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.881	8.773	V_x (kip)	-2.369	-3.952	V_z (kip)	-0.123	-0.184	M_x (kipft)	-0.393	-0.577	M_z (kipft)	26.390	44.487	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.369 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.78967 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(26.39 \text{ kipft}) + ((-2.369 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.7967 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.9922 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.123 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.041 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.393 \text{ kipft}) + ((-0.123 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.131 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0454 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9922 \text{ ft}), (2.0454 \text{ ft})]$$

$$L_{e,req} = 7.992 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.992 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.888$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(5.881 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.83199 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.83199 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.416$$

Status: **PASS**
Ratio: **0.420**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.78967 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.7967 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7967 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (8.7967 \text{ kipft/ft})) + (4 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2625 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (8.7967 \text{ kipft/ft})) + (3 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (8.7967 \text{ kipft/ft})) + (2 \times (-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.22963 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (8.7967 \text{ kipft/ft})) + ((-0.78967 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2202 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2625 \text{ ft})}{2}$$

$$p_a = 0.46969 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22963 \text{ kip/ft}^2)}{(0.46969 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.48891$$

Status: **PASS**
Ratio: **0.490**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2202 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90383$$

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.041 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.131 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.131 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.041 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.131 \text{ kipft/ft})) + (4 \times (-0.041 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4894 \text{ ft}$$

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

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

$$p = \frac{1.178 [(4 \times (0.131 \text{ kipft/ft})) + (3 \times (-0.041 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 [(3 \times (0.131 \text{ kipft/ft})) + (2 \times (-0.041 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = -0.014328 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 [(2 \times (0.131 \text{ kipft/ft})) + ((-0.041 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = -0.01245 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4894 \text{ ft})}{2}$$

$$p_a = 0.4867 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.014328 \text{ kip/ft}^2)}{(0.4867 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.029438$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

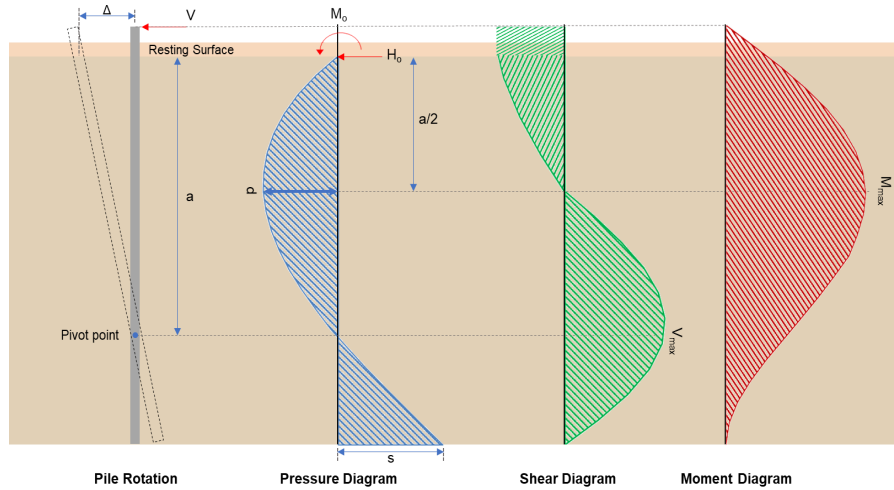
Status: **PASS**
Ratio: **-0.030**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(-0.01245 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = -0.0092225$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.952 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3173 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.487 \text{ kipft}) + ((-3.952 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.829 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.829 \text{ kipft/ft})}{(-1.3173 \text{ kip/ft})}$$

$$E = 11.257 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.829 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3173 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (14.829 \text{ kipft/ft})) + (4 \times (-1.3173 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2608 \text{ ft}$$

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

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

$$V_{max} = ((-1.3173 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2608 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2608 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.353 \text{ kip}$$

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

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

$$M_{max} = ((-1.3173 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(11.257 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (11.257 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 47.875 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.184 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.061333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.577 \text{ kipft}) + ((-0.184 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19233 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19233 \text{ kipft/ft})}{(-0.061333 \text{ kip/ft})}$$

$$E = 3.1359 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19233 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.061333 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.19233 \text{ kipft/ft})) + (4 \times (-0.061333 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4926 \text{ ft}$$

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

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

$$V_{max} = ((-0.061333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4926 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4926 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23672 \text{ kip}$$

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

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

$$M_{max} = ((-0.061333 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.1359 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.1359 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4926 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.91823 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.773 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.099 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.099 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

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

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.773 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0069965$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.773 \text{ kip} \rightarrow 8773 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8773 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.927 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

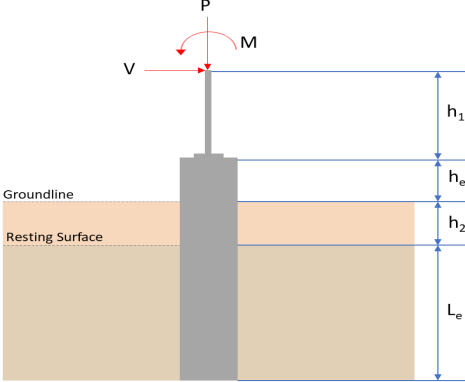
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.927 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 75.927 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.927 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.163 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.353 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(11.353 \text{ kip})}{(74.163 \text{ kip})}$ $Ratio = 0.15308$ <p>Considering z-direction:</p> <p>$V_{max} = 0.23672 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.23672 \text{ kip})}{(74.163 \text{ kip})}$ $Ratio = 0.0031919$	<p>Status: PASS Ratio: 0.150</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 47.875 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(47.875 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.77184$	<p>Status: PASS Ratio: 0.770</p>
	<p>Considering z-direction: $M_{max} = 0.91823 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.91823 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.014804$	<p>Status: PASS Ratio: 0.010</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: round $D = 36$ in - Pile diameter $L = 9.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.679</td> <td>9.983</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.688</td> <td>-4.481</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.012</td> <td>-0.022</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.037</td> <td>-0.064</td> </tr> <tr> <td>M_z (kipft)</td> <td>29.948</td> <td>50.637</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.679	9.983	V_x (kip)	-2.688	-4.481	V_z (kip)	-0.012	-0.022	M_x (kipft)	-0.037	-0.064	M_z (kipft)	29.948	50.637	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.688 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.896 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(29.948 \text{ kipft}) + ((-2.688 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 9.9827 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2367 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.037 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.012333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0489 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.2367 \text{ ft}), (1.0489 \text{ ft})]$$

$$L_{e,req} = 8.237 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.237 \text{ ft})}{(9.25 \text{ ft})}$$

$$\text{Ratio} = 0.89049$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(6.679 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.94489 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.94489 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.47244$$

Status: **PASS**
Ratio: **0.470**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3.0833$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.896 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.9827 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.9827 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (9.9827 \text{ kipft/ft})) + (4 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4413 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (9.9827 \text{ kipft/ft})) + (3 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (9.9827 \text{ kipft/ft})) + (2 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$$

$$p = 0.23372 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (9.9827 \text{ kipft/ft})) + ((-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$$

$$s = 1.2863 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4413 \text{ ft})}{2}$$

$$p_a = 0.4831 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23372 \text{ kip/ft}^2)}{(0.4831 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.4838$$

Status: **PASS**
Ratio: **0.480**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$$

$$p_s = 1.3875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2863 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92706$$

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

Considering z-direction:

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

$M_o = 0.012333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012333 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.012333 \text{ kipft/ft})) + (4 \times (-0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.6806 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.012333 \text{ kipft/ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (0.012333 \text{ kipft/ft})) + (2 \times (-0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$$

$$p = -0.001415 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.012333 \text{ kipft/ft})) + ((-0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$$

$$s = -0.0013586 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.6806 \text{ ft})}{2}$$

$$p_a = 0.50104 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.001415 \text{ kip/ft}^2)}{(0.50104 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0028241$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$$

$$p_s = 1.3875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

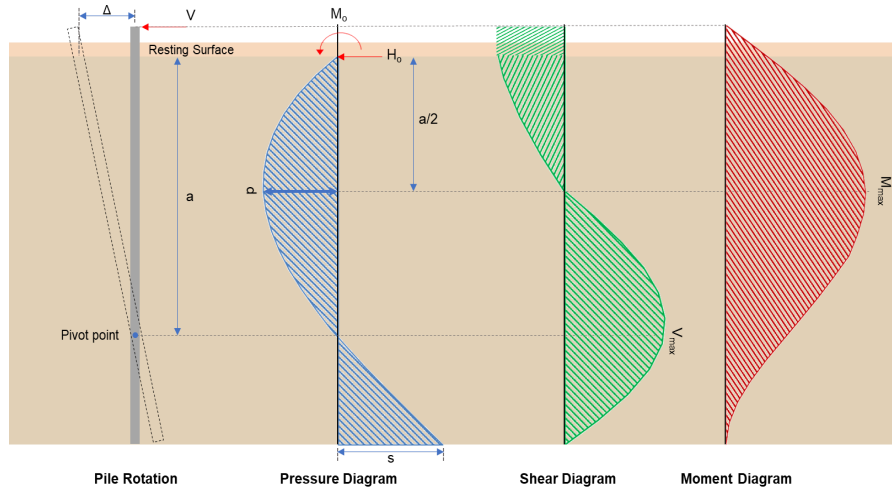
Status: **PASS**
Ratio: **0.000**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(-0.0013586 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00097914$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.481 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.4937 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(50.637 \text{ kipft}) + ((-4.481 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 16.879 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.879 \text{ kipft/ft})}{(-1.4937 \text{ kip/ft})}$$

$$E = 11.3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.879 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.4937 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (16.879 \text{ kipft/ft})) + (4 \times (-1.4937 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4388 \text{ ft}$$

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

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

$$V_{max} = ((-1.4937 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4388 \text{ ft})}{(9.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4388 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.643 \text{ kip}$$

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

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

$$M_{max} = ((-1.4937 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(11.3 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 54.727 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.022 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0073333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((-0.022 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.021333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.021333 \text{ kipft/ft})}{(-0.0073333 \text{ kip/ft})}$$

$$E = 2.9091 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021333 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.0073333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.021333 \text{ kipft/ft})) + (4 \times (-0.0073333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.6904 \text{ ft}$$

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

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

$$V_{max} = ((-0.0073333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.6904 \text{ ft})}{(9.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.6904 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.027006 \text{ kip}$$

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

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

$$M_{max} = ((-0.0073333 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(2.9091 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.10686 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.983 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.061 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.061 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

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

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.983 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0079615$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.983 \text{ kip} \rightarrow 9983 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9983 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.133 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

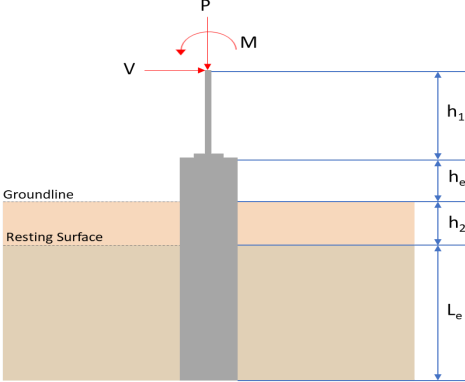
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.133 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 76.133 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.133 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.297 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 12.643 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(12.643 \text{ kip})}{(74.297 \text{ kip})}$ $Ratio = 0.17016$ <p>Considering z-direction:</p> <p>$V_{max} = 0.027006 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.027006 \text{ kip})}{(74.297 \text{ kip})}$ $Ratio = 0.00036349$	<p>Status: PASS Ratio: 0.170</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 54.727 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(54.727 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.88231$	<p>Status: PASS Ratio: 0.880</p>
	<p>Considering z-direction: $M_{max} = 0.10686 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.10686 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0017227$	<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: round $D = 36$ in - Pile diameter $L = 9.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.679</td> <td>9.983</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.688</td> <td>-4.481</td> </tr> <tr> <td>V_z (kip)</td> <td>0.012</td> <td>0.022</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.037</td> <td>0.064</td> </tr> <tr> <td>M_z (kipft)</td> <td>29.948</td> <td>50.637</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	6.679	9.983	V_x (kip)	-2.688	-4.481	V_z (kip)	0.012	0.022	M_x (kipft)	0.037	0.064	M_z (kipft)	29.948	50.637	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.688 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.896 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(29.948 \text{ kipft}) + ((-2.688 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 9.9827 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2367 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.037 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.012333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2655 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.2367 \text{ ft}), (1.2655 \text{ ft})]$$

$$L_{e,req} = 8.237 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (9.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.237 \text{ ft})}{(9.25 \text{ ft})}$$

$$\text{Ratio} = 0.89049$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(6.679 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.94489 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.94489 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.47244$$

Status: **PASS**
Ratio: **0.470**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3.0833$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.896 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.9827 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.9827 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (9.9827 \text{ kipft/ft})) + (4 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4413 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (9.9827 \text{ kipft/ft})) + (3 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (9.9827 \text{ kipft/ft})) + (2 \times (-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$$

$$p = 0.23372 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (9.9827 \text{ kipft/ft})) + ((-0.896 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$$

$$s = 1.2863 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4413 \text{ ft})}{2}$$

$$p_a = 0.4831 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23372 \text{ kip/ft}^2)}{(0.4831 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.4838$$

Status: **PASS**
Ratio: **0.480**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$$

$$p_s = 1.3875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2863 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92706$$

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

Considering z-direction:

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

$M_o = 0.012333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012333 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (0.004 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.012333 \text{ kipft/ft})) + (4 \times (0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.6806 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.012333 \text{ kipft/ft})) + (3 \times (0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]^2}{(9.25 \text{ ft})^2 \times [(3 \times (0.012333 \text{ kipft/ft})) + (2 \times (0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]}$$

$$p = 0.0031885 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.012333 \text{ kipft/ft})) + ((0.004 \text{ kip/ft}) \times (9.25 \text{ ft}))]}{(9.25 \text{ ft})^2}$$

$$s = 0.0067928 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.6806 \text{ ft})}{2}$$

$$p_a = 0.50104 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0031885 \text{ kip/ft}^2)}{(0.50104 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0063637$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.25 \text{ ft})$$

$$p_s = 1.3875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

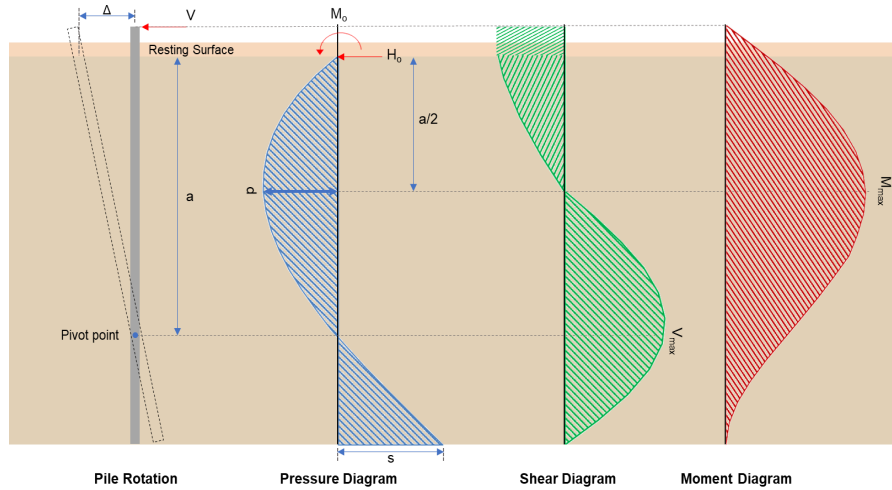
Status: **PASS**
Ratio: **0.010**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0067928 \text{ kip/ft}^2)}{(1.3875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0048957$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.481 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.4937 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(50.637 \text{ kipft}) + ((-4.481 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 16.879 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.879 \text{ kipft/ft})}{(-1.4937 \text{ kip/ft})}$$

$$E = 11.3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.879 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (-1.4937 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (16.879 \text{ kipft/ft})) + (4 \times (-1.4937 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.4388 \text{ ft}$$

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

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

$$V_{max} = ((-1.4937 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4388 \text{ ft})}{(9.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4388 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.643 \text{ kip}$$

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

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

$$M_{max} = ((-1.4937 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(11.3 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.3 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.4388 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 54.727 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.022 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0073333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((0.022 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.021333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.021333 \text{ kipft/ft})}{(0.0073333 \text{ kip/ft})}$$

$$E = 2.9091 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021333 \text{ kipft/ft}) \times (9.25 \text{ ft})) + (3 \times (0.0073333 \text{ kip/ft}) \times (9.25 \text{ ft})^2)}{(6 \times (0.021333 \text{ kipft/ft})) + (4 \times (0.0073333 \text{ kip/ft}) \times (9.25 \text{ ft}))}$$

$$a = 6.6904 \text{ ft}$$

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

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

$$V_{max} = ((0.0073333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.6904 \text{ ft})}{(9.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.6904 \text{ ft})}{(9.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.027006 \text{ kip}$$

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

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

$$M_{max} = ((0.0073333 \text{ kip/ft}) \times (36 \text{ in}) \times (9.25 \text{ ft})) \times \left[\left(\frac{(2.9091 \text{ ft})}{(9.25 \text{ ft})} + \frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 3 \right) \times \left(\frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9091 \text{ ft})}{(9.25 \text{ ft})} + 2 \right) \times \left(\frac{(6.6904 \text{ ft})}{2 \times (9.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.10686 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.983 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.061 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.061 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

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

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.983 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0079615$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.983 \text{ kip} \rightarrow 9983 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9983 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.133 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.133 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 76.133 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.133 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.297 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 12.643 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(12.643 \text{ kip})}{(74.297 \text{ kip})}$ $Ratio = 0.17016$ <p>Considering z-direction:</p> <p>$V_{max} = 0.027006 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.027006 \text{ kip})}{(74.297 \text{ kip})}$ $Ratio = 0.00036349$	<p>Status: PASS Ratio: 0.170</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 54.727 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(54.727 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.88231$	<p>Status: PASS Ratio: 0.880</p>
	<p>Considering z-direction: $M_{max} = 0.10686 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.10686 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0017227$	<p>Status: PASS Ratio: 0.000</p>