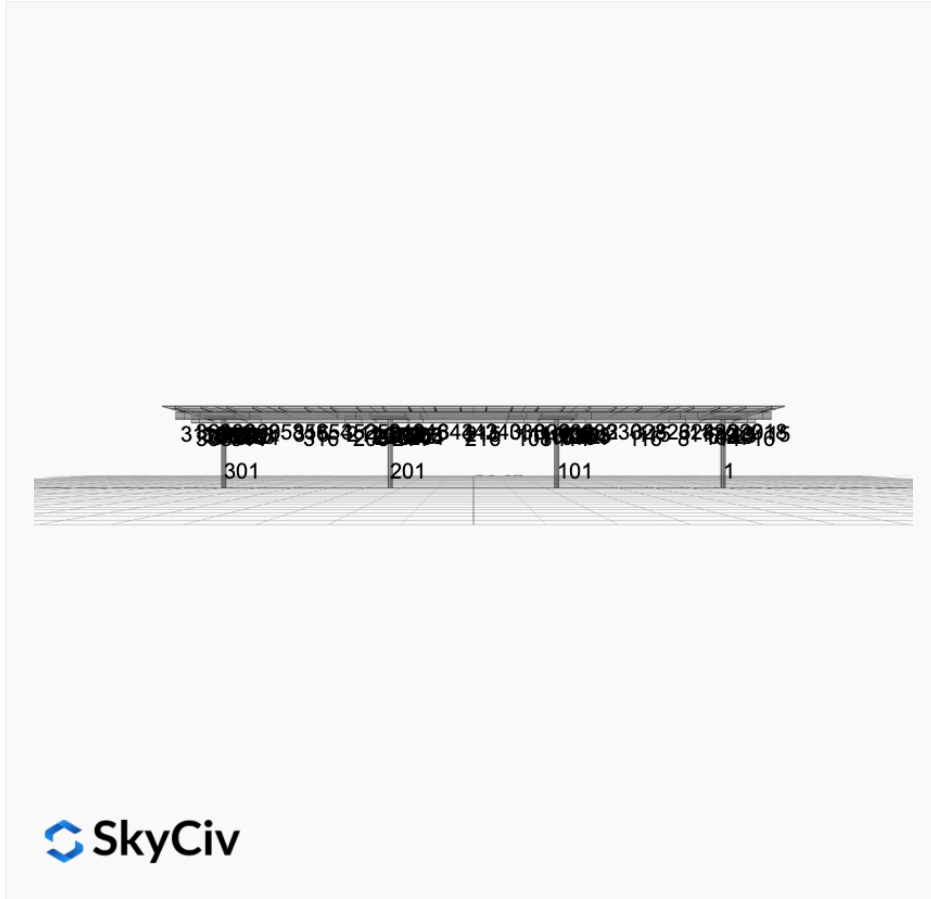


Project Details



Project Name: Mort IA 5x12 36inRound - V1Jb **Date:** Fri Apr 18 2025
Location: 103 Enterprise Dr, Northwood, IA 50459, USA **Number of Modules:** 60
Unique ID: 4P-19.75-6TOP-XD-12-L-5Hx12W-198E **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.83 ft
Array Dimensions E/W	68.80 ft
Winter Tilt Angle	2
Front Edge Clearance	8 ft

MT Solar Bill of Materials (4P-19.75-6TOP-XD-12-L-5Hx12W-198E)

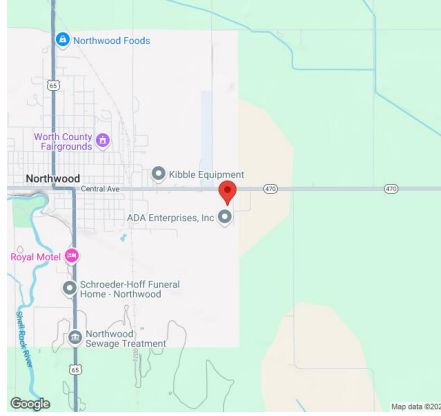
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

Rail Bill of Materials

Part	Qty
Rails (226in)	24
Rail Attachment	96

Part	Qty
Module Mid Clamp	96
Module End Clamp	48
Ground Lug	12

Site Details:



Site Address: 103 Enterprise Dr, Northwood, IA 50459, USA

Array Specification

Duty Classification:	XD
Module Width:	44.70 in
Module Length:	67.80in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Winter Tilt Angle:	2
Front Edge Clearance:	8
Total Array Height at Tilt:	8.66 ft
Total Frame Length:	68.75 ft
Module Info/Notes:	410w
Array Dimensions N/S:	18.83 ft
Array Dimensions E/W:	68.80 ft
Rail Length:	226.00 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.33 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: 5.75 ft Pile 2: 6.00 ft Pile 3: 6.00 ft Pile 4: 5.75 ft
Foundation Volume:	6.152 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	103 Enterprise Dr, Northwood, IA 50459, USA
Wind Speed:	103 mph

Snow Load:

40 psf

Design Disclaimer

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

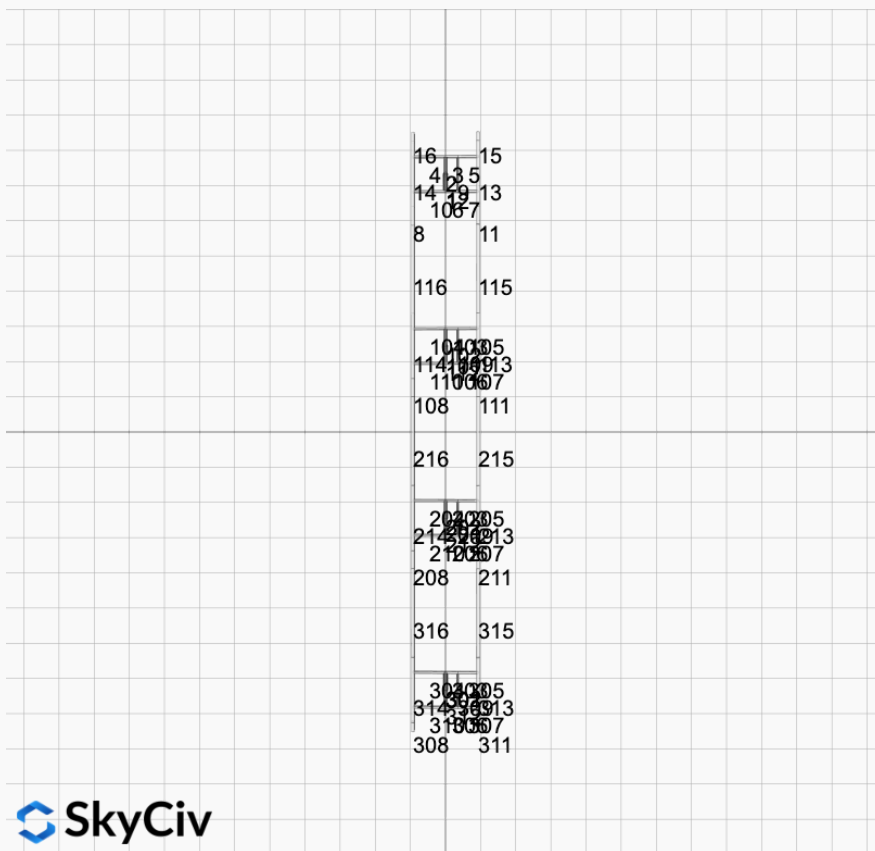
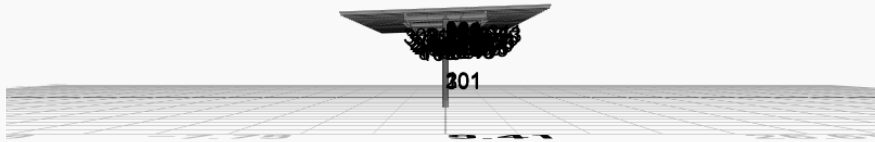
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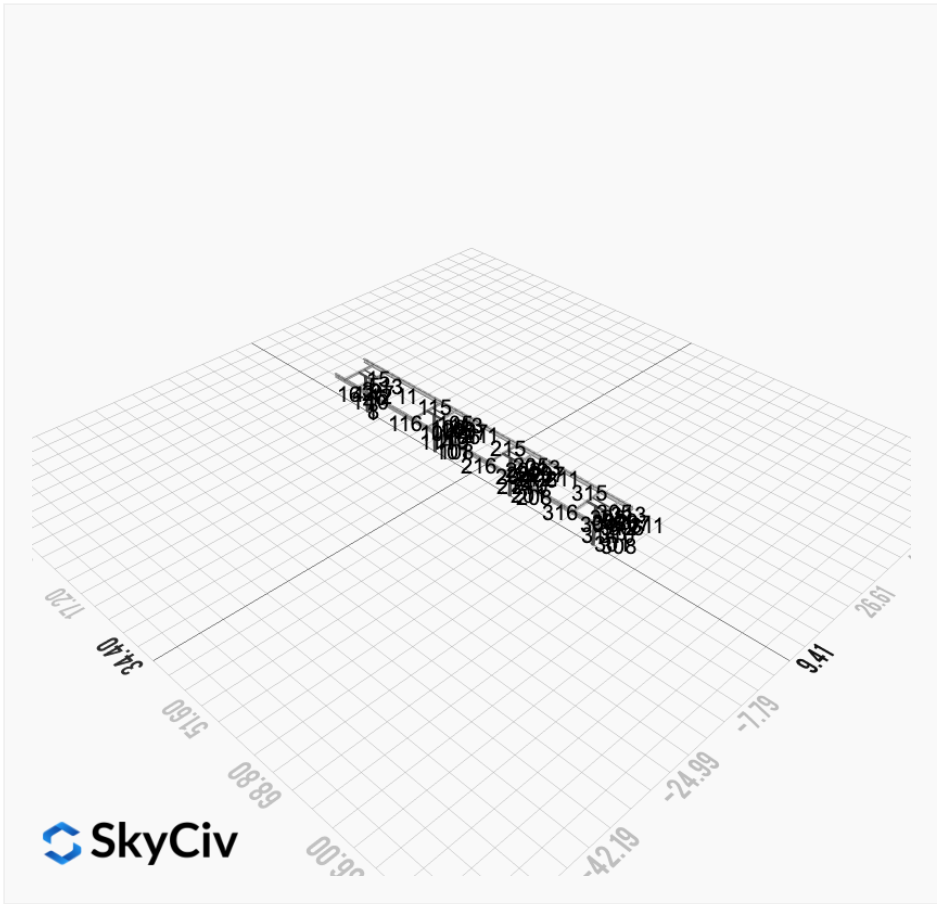
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Design Notes:

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

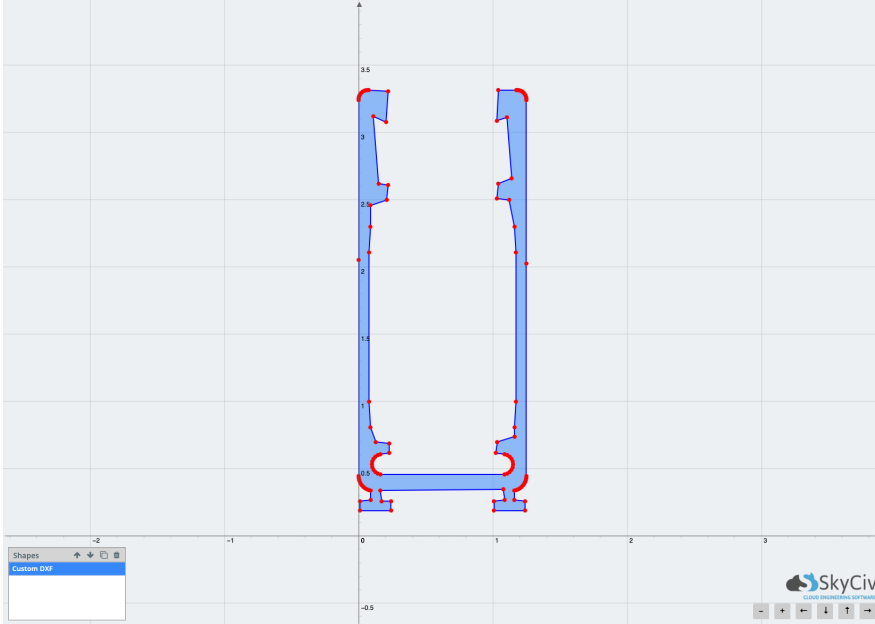






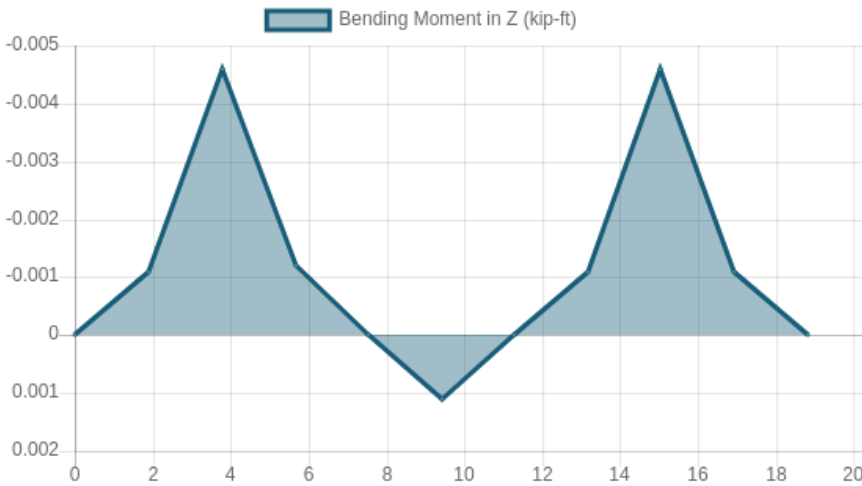
Rail Design Check

Rail Length: 18.833333333333332 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 2.866666666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0693 kip/ft
Snow (Y): -0.0024 kip/ft
Wind uplift Case A: 0.0014 kip/ft
Wind uplift Case A: 0.0014 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0365 kip/ft

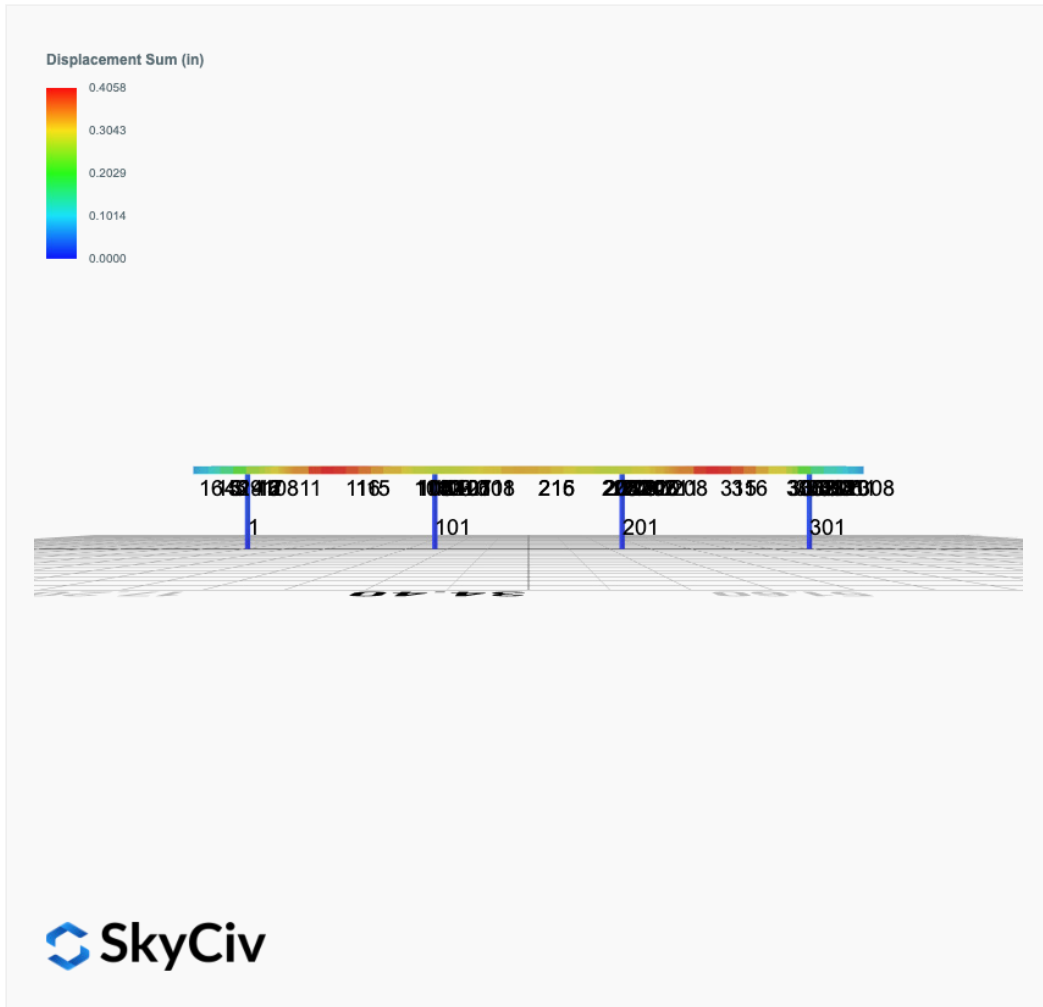


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	5.75490705	0.167	PASS
Material Yield	34.5	5.75490705	0.167	PASS
Material Strength	37	5.75490705	0.156	PASS

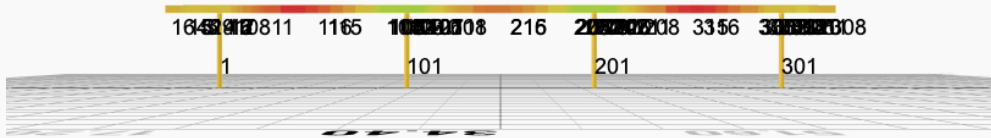
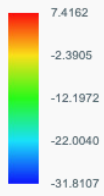
Member 1, ULS: 1.1.4D



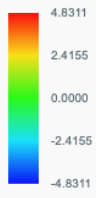
FEM Results (Envelope Worst Case for each member)



Top Bending Stress Z (ksi)



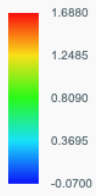
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0035	1.9259	0.1070	0.2622	-0.0060	0.0075
ULS: 2. D + L	0.0035	1.9259	0.1070	0.2622	-0.0060	0.0075
ULS: 3. D + (S or Lr or R)	0.0178	8.1241	0.5522	1.3546	-0.0310	-0.0933
ULS: 3. D + (S or Lr or R)	0.0035	1.9259	0.1070	0.2622	-0.0060	0.0075
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0142	6.5745	0.4409	1.0815	-0.0247	-0.0681
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	1.9259	0.1070	0.2622	-0.0060	0.0075
ULS: 5b. D + 0.7E	0.0035	1.9259	0.1070	0.2622	-0.0060	0.0075
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0142	6.5745	0.4409	1.0815	-0.0247	-0.0681
ULS: 8. 0.6D + 0.7E	0.0021	1.1555	0.0642	0.1573	-0.0036	0.0045
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0412	3.3687	0.2090	0.5119	-0.0143	-1.3161
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0412	3.3687	0.2090	0.5119	-0.0143	-1.3161
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0276	2.4858	0.1495	0.3655	-0.0156	2.8719
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0562	0.8842	0.0293	0.0731	0.0094	-4.1255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0192	7.6567	0.5174	1.2688	-0.0310	-1.0609
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0192	7.6567	0.5174	1.2688	-0.0310	-1.0609
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0090	6.9945	0.4728	1.1590	-0.0319	2.0801
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0538	5.7933	0.3826	0.9397	-0.0132	-3.1679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0300	3.0080	0.1835	0.4495	-0.0123	-0.9852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0300	3.0080	0.1835	0.4495	-0.0123	-0.9852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0198	2.3458	0.1389	0.3397	-0.0132	2.1558
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0430	1.1447	0.0487	0.1204	0.0056	-3.0923
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0426	2.5984	0.1662	0.4070	-0.0119	-1.3192
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0426	2.5984	0.1662	0.4070	-0.0119	-1.3192
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0290	1.7155	0.1067	0.2606	-0.0132	2.8689
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0548	0.1139	-0.0136	-0.0318	0.0118	-4.1285

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	13.4295
Shear X	-0.1007
Shear Z	0.9292
Moment X	2.2852
Moment Y (Twist)	0.0560
Moment Z	7.1810

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.1241
Shear X	-0.0562
Shear Z	0.5522
Moment X	1.3546
Moment Y (Twist)	0.0319
Moment Z	4.1285

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	-0.0035	2.7114	-0.0175	-0.0436	0.0018	0.0575
ULS: 2. D + L	-0.0035	2.7114	-0.0175	-0.0436	0.0018	0.0575
ULS: 3. D + (S or Lr or R)	-0.0178	12.1654	-0.0905	-0.2261	0.0092	0.1694
ULS: 3. D + (S or Lr or R)	-0.0035	2.7114	-0.0175	-0.0436	0.0018	0.0575
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0142	9.8019	-0.0723	-0.1805	0.0073	0.1414

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.7114	-0.0175	-0.0436	0.0018	0.0575
ULS: 5b. D + 0.7E	-0.0035	2.7114	-0.0175	-0.0436	0.0018	0.0575
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0142	9.8019	-0.0723	-0.1805	0.0073	0.1414
ULS: 8. 0.6D + 0.7E	-0.0021	1.6268	-0.0105	-0.0262	0.0011	0.0345
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0861	4.9143	-0.0351	-0.0871	0.0060	-1.5518
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0861	4.9143	-0.0351	-0.0871	0.0060	-1.5518
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0217	3.5623	-0.0230	-0.0578	-0.0024	3.6778
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0355	1.1270	-0.0068	-0.0161	0.0073	-5.1489
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0762	11.4541	-0.0854	-0.2131	0.0105	-1.0656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0762	11.4541	-0.0854	-0.2131	0.0105	-1.0656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0279	10.4401	-0.0764	-0.1911	0.0042	2.8566
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0150	8.6136	-0.0642	-0.1598	0.0115	-3.7634
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0655	4.3636	-0.0307	-0.0762	0.0049	-1.1495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0655	4.3636	-0.0307	-0.0762	0.0049	-1.1495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0171	3.3496	-0.0216	-0.0543	-0.0013	2.7727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0258	1.5231	-0.0094	-0.0229	0.0059	-3.8473
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0848	3.8297	-0.0281	-0.0697	0.0053	-1.5748
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0848	3.8297	-0.0281	-0.0697	0.0053	-1.5748
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0203	2.4777	-0.0160	-0.0404	-0.0031	3.6548
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0369	0.0424	0.0002	0.0014	0.0066	-5.1719

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	20.2169
Shear X	-0.1497
Shear Z	-0.1536
Moment X	-0.3847
Moment Y (Twist)	0.0196
Moment Z	8.9525

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	12.1654
Shear X	-0.0861
Shear Z	-0.0905
Moment X	-0.2261
Moment Y (Twist)	0.0115
Moment Z	5.1719

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	-0.0035	2.7114	0.0175	0.0436	-0.0018	0.0575
ULS: 2. D + L	-0.0035	2.7114	0.0175	0.0436	-0.0018	0.0575
ULS: 3. D + (S or Lr or R)	-0.0178	12.1654	0.0905	0.2261	-0.0091	0.1694
ULS: 3. D + (S or Lr or R)	-0.0035	2.7114	0.0175	0.0436	-0.0018	0.0575
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0142	9.8019	0.0723	0.1805	-0.0073	0.1414
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0035	2.7114	0.0175	0.0436	-0.0018	0.0575
ULS: 5b. D + 0.7E	-0.0035	2.7114	0.0175	0.0436	-0.0018	0.0575
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0142	9.8019	0.0723	0.1805	-0.0073	0.1414
ULS: 8. 0.6D + 0.7E	-0.0021	1.6268	0.0105	0.0262	-0.0011	0.0345
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0861	4.9143	0.0351	0.0871	-0.0060	-1.5518
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0861	4.9143	0.0351	0.0871	-0.0060	-1.5518
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0217	3.5623	0.0230	0.0578	0.0024	3.6778
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0355	1.1270	0.0068	0.0161	-0.0073	-5.1489

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0762	11.4541	0.0854	0.2131	-0.0105	-1.0656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0762	11.4541	0.0854	0.2131	-0.0105	-1.0656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0279	10.4401	0.0764	0.1911	-0.0042	2.8566
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0150	8.6136	0.0642	0.1598	-0.0115	-3.7634
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0655	4.3636	0.0307	0.0762	-0.0049	-1.1495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0655	4.3636	0.0307	0.0762	-0.0049	-1.1495
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0171	3.3496	0.0216	0.0543	0.0013	2.7727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0258	1.5231	0.0094	0.0229	-0.0059	-3.8473
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0848	3.8297	0.0281	0.0697	-0.0053	-1.5748
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0848	3.8297	0.0281	0.0697	-0.0053	-1.5748
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0203	2.4777	0.0160	0.0404	0.0031	3.6548
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0369	0.0424	-0.0002	-0.0014	-0.0066	-5.1719

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	20.2169
Shear X	-0.1497
Shear Z	0.1536
Moment X	0.3847
Moment Y (Twist)	0.0194
Moment Z	8.9525

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	12.1654
Shear X	-0.0861
Shear Z	0.0905
Moment X	0.2261
Moment Y (Twist)	0.0115
Moment Z	5.1719

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	0.0035	1.9259	-0.1070	-0.2622	0.0060	0.0075
ULS: 2. D + L	0.0035	1.9259	-0.1070	-0.2622	0.0060	0.0075
ULS: 3. D + (S or Lr or R)	0.0178	8.1241	-0.5522	-1.3547	0.0310	-0.0933
ULS: 3. D + (S or Lr or R)	0.0035	1.9259	-0.1070	-0.2622	0.0060	0.0075
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0142	6.5745	-0.4409	-1.0815	0.0248	-0.0681
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0035	1.9259	-0.1070	-0.2622	0.0060	0.0075
ULS: 5b. D + 0.7E	0.0035	1.9259	-0.1070	-0.2622	0.0060	0.0075
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0142	6.5745	-0.4409	-1.0815	0.0248	-0.0681
ULS: 8. 0.6D + 0.7E	0.0021	1.1555	-0.0642	-0.1573	0.0036	0.0045
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0412	3.3687	-0.2090	-0.5119	0.0143	-1.3161
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0412	3.3687	-0.2090	-0.5119	0.0143	-1.3161
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0276	2.4858	-0.1495	-0.3655	0.0156	2.8719
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0562	0.8842	-0.0293	-0.0731	-0.0094	-4.1255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0192	7.6567	-0.5174	-1.2688	0.0310	-1.0609
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0192	7.6567	-0.5174	-1.2688	0.0310	-1.0609
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0090	6.9945	-0.4728	-1.1590	0.0319	2.0802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0538	5.7933	-0.3826	-0.9397	0.0132	-3.1679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0300	3.0080	-0.1835	-0.4495	0.0123	-0.9852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0300	3.0080	-0.1835	-0.4495	0.0123	-0.9852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0198	2.3458	-0.1389	-0.3397	0.0132	2.1558
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0430	1.1447	-0.0487	-0.1204	-0.0056	-3.0923

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0426	2.5984	-0.1662	-0.4070	0.0119	-1.3192
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0426	2.5984	-0.1662	-0.4070	0.0119	-1.3192
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0290	1.7155	-0.1067	-0.2606	0.0132	2.8689
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0548	0.1139	0.0136	0.0318	-0.0118	-4.1285

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	13.4295
Shear X	-0.1007
Shear Z	-0.9292
Moment X	-2.2852
Moment Y (Twist)	0.0561
Moment Z	7.1810

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.1241
Shear X	-0.0562
Shear Z	-0.5522
Moment X	-1.3547
Moment Y (Twist)	0.0319
Moment Z	4.1285

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: Mort IA 5x12 36inRound - 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)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
7	6in Pipe Sch 40	6.63	0.28				

ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

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

212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.20	6.46	56.26	44.91
214	159.30	97.43	31.89	6.46	56.26	44.91
215	159.30	75.13	21.26	6.46	56.26	44.91
216	159.30	75.13	21.08	6.46	56.26	44.91
301	251.16	132.60	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	137.23	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	137.23	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	45.10	6.46	56.26	44.91
314	159.30	97.43	44.91	6.46	56.26	44.91
315	159.30	75.13	20.71	6.46	56.26	44.91
316	159.30	75.13	20.70	6.46	56.26	44.91

Design Ratio

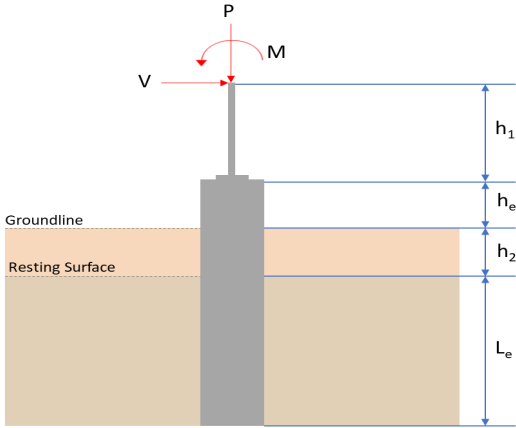
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.101	0.170	0.128	0.001	0.012	0.222	#23	0.467	Not Required	Pass
2	0.002	0.316	0.003	0.074	0.000	0.320	#21	0.036	Not Required	Pass
3	0.002	0.506	0.032	0.050	0.008	0.536	#23	0.046	Not Required	Pass
4	0.001	0.526	0.041	0.053	0.008	0.567	#21	0.122	Not Required	Pass
5	0.001	0.314	0.017	0.050	0.007	0.331	#23	0.076	Not Required	Pass
6	0.000	0.692	0.043	0.071	0.011	0.732	#23	0.046	Not Required	Pass
7	0.000	0.428	0.038	0.069	0.012	0.448	#23	0.076	Not Required	Pass
8	0.003	0.183	0.020	0.044	0.002	0.194	#21	0.102	Not Required	Pass
9	0.001	0.090	0.037	0.004	0.003	0.127	#21	0.206	Not Required	Pass
10	0.002	0.717	0.027	0.072	0.005	0.745	#21	0.082	Not Required	Pass
11	0.004	0.173	0.029	0.043	0.002	0.181	#23	0.102	Not Required	Pass
12	0.002	0.516	0.008	0.102	0.001	0.522	#21	0.054	Not Required	Pass
13	0.006	0.091	0.060	0.057	0.003	0.119	#21	0.306	Not Required	Pass
14	0.004	0.095	0.057	0.060	0.004	0.111	#21	0.204	Not Required	Pass
15	0.000	0.005	0.001	0.008	0.000	0.006	#23	Not Required	Not Required	Pass
16	0.000	0.005	0.001	0.009	0.000	0.006	#21	Not Required	Not Required	Pass
101	0.152	0.212	0.021	0.002	0.002	0.234	#16	0.467	Not Required	Pass
102	0.000	0.661	0.010	0.135	0.001	0.670	#21	0.054	Not Required	Pass
103	0.001	0.917	0.012	0.092	0.002	0.929	#23	0.046	Not Required	Pass
104	0.001	0.958	0.017	0.096	0.003	0.963	#21	0.122	Not Required	Warn
105	0.001	0.568	0.023	0.091	0.006	0.576	#23	0.076	Not Required	Pass
106	0.001	0.891	0.003	0.089	0.001	0.892	#23	0.046	Not Required	Pass
107	0.001	0.553	0.014	0.089	0.003	0.555	#23	0.076	Not Required	Pass
108	0.002	0.062	0.018	0.053	0.002	0.081	#21	0.102	Not Required	Pass
109	0.002	0.102	0.010	0.001	0.001	0.112	#21	0.206	Not Required	Pass
110	0.001	0.028	0.021	0.002	0.005	0.047	#21	0.092	Not Required	Pass

110	0.001	0.920	0.021	0.093	0.003	0.947	#21	0.002	Not Required	Pass
111	0.003	0.062	0.018	0.051	0.002	0.081	#21	0.102	Not Required	Pass
112	0.001	0.628	0.009	0.131	0.001	0.637	#21	0.054	Not Required	Pass
113	0.006	0.295	0.054	0.073	0.003	0.314	#21	0.306	Not Required	Pass
114	0.005	0.308	0.057	0.077	0.003	0.322	#21	0.306	Not Required	Pass
115	0.007	0.463	0.024	0.059	0.002	0.488	#23	0.507	Not Required	Pass
116	0.005	0.483	0.028	0.062	0.002	0.511	#21	0.507	Not Required	Pass
201	0.152	0.212	0.021	0.002	0.002	0.234	#16	0.467	Not Required	Pass
202	0.001	0.628	0.009	0.131	0.001	0.637	#21	0.054	Not Required	Pass
203	0.001	0.891	0.003	0.089	0.001	0.892	#23	0.046	Not Required	Pass
204	0.001	0.928	0.021	0.093	0.005	0.947	#21	0.082	Not Required	Pass
205	0.001	0.553	0.014	0.089	0.003	0.555	#23	0.076	Not Required	Pass
206	0.001	0.917	0.012	0.092	0.002	0.929	#23	0.046	Not Required	Pass
207	0.001	0.568	0.023	0.091	0.006	0.576	#23	0.076	Not Required	Pass
208	0.003	0.098	0.026	0.062	0.002	0.107	#21	0.102	Not Required	Pass
209	0.002	0.102	0.010	0.001	0.001	0.112	#21	0.206	Not Required	Pass
210	0.001	0.958	0.017	0.096	0.003	0.963	#21	0.122	Not Required	Warn
211	0.004	0.096	0.024	0.059	0.002	0.099	#23	0.102	Not Required	Pass
212	0.000	0.661	0.010	0.135	0.001	0.670	#21	0.054	Not Required	Pass
213	0.006	0.295	0.054	0.073	0.003	0.314	#21	0.306	Not Required	Pass
214	0.005	0.308	0.057	0.077	0.003	0.322	#21	0.306	Not Required	Pass
215	0.006	0.269	0.026	0.051	0.002	0.298	#23	0.507	Not Required	Pass
216	0.004	0.274	0.025	0.053	0.002	0.301	#21	0.507	Not Required	Pass
301	0.101	0.170	0.128	0.001	0.012	0.222	#23	0.467	Not Required	Pass
302	0.002	0.516	0.008	0.102	0.001	0.522	#21	0.054	Not Required	Pass
303	0.000	0.692	0.043	0.071	0.011	0.732	#23	0.046	Not Required	Pass
304	0.002	0.717	0.027	0.072	0.005	0.745	#21	0.082	Not Required	Pass
305	0.000	0.428	0.038	0.069	0.012	0.448	#23	0.076	Not Required	Pass
306	0.002	0.506	0.032	0.050	0.008	0.536	#23	0.046	Not Required	Pass
307	0.001	0.314	0.017	0.050	0.007	0.331	#23	0.076	Not Required	Pass
308	0.000	0.005	0.001	0.009	0.000	0.006	#21	Not Required	Not Required	Pass
309	0.001	0.090	0.037	0.004	0.003	0.127	#21	0.206	Not Required	Pass
310	0.001	0.526	0.041	0.053	0.008	0.567	#21	0.122	Not Required	Pass
311	0.000	0.005	0.001	0.008	0.000	0.006	#23	Not Required	Not Required	Pass
312	0.002	0.316	0.003	0.074	0.000	0.320	#21	0.036	Not Required	Pass
313	0.006	0.091	0.060	0.057	0.003	0.119	#21	0.204	Not Required	Pass
314	0.004	0.095	0.057	0.060	0.004	0.111	#21	0.306	Not Required	Pass
315	0.007	0.501	0.029	0.043	0.002	0.528	#23	0.507	Not Required	Pass
316	0.005	0.525	0.028	0.044	0.002	0.555	#21	0.507	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</p> <p>Pile shape: round $D = 36$ in - Pile diameter $L = 5.75$ 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="368 1061 1227 1162"> <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="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.124</td> <td>13.430</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.056</td> <td>-0.101</td> </tr> <tr> <td>V_z (kip)</td> <td>0.552</td> <td>0.929</td> </tr> <tr> <td>M_x (kipft)</td> <td>1.355</td> <td>2.285</td> </tr> <tr> <td>M_z (kipft)</td> <td>4.129</td> <td>7.181</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	8.124	13.430	V_x (kip)	-0.056	-0.101	V_z (kip)	0.552	0.929	M_x (kipft)	1.355	2.285	M_z (kipft)	4.129	7.181	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
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M_x (kipft)	1.355	2.285																										
M_z (kipft)	4.129	7.181																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.056 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.018667 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(4.129 \text{ kipft}) + ((-0.056 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.3763 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.4663 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.552 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.184 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.355 \text{ kipft}) + ((0.552 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.45167 \text{ kipft/ft}$$

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:

$L_{e,z} = 5.2972 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4663 \text{ ft}), (5.2972 \text{ ft})]$$

$$L_{e,req} = 5.466 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.466 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95061$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(8.124 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.1493 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.1493 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.57466$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 1.9167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.018667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.3763 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.3763 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (1.3763 \text{ kipft/ft})) + (4 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.857 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.3763 \text{ kipft/ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (1.3763 \text{ kipft/ft})) + (2 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24455 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (1.3763 \text{ kipft/ft})) + ((-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.7541 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.857 \text{ ft})}{2}$$

$$p_a = 0.28928 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24455 \text{ kip/ft}^2)}{(0.28928 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84539$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.7541 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87431$$

Status: **PASS**
Ratio: **0.850**

Status: **PASS**
Ratio: **0.870**

Considering z-direction:

$H_o = 0.184 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.45167 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.45167 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.184 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.45167 \text{ kipft/ft})) + (4 \times (0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1254 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.45167 \text{ kipft/ft})) + (3 \times (0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.45167 \text{ kipft/ft})) + (2 \times (0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.25464 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.45167 \text{ kipft/ft})) + ((0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.55911 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1254 \text{ ft})}{2}$$

$$p_a = 0.30941 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25464 \text{ kip/ft}^2)}{(0.30941 \text{ kip/ft}^2)}$$

(0.0051 kip/ft)

$$Ratio = 0.823$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

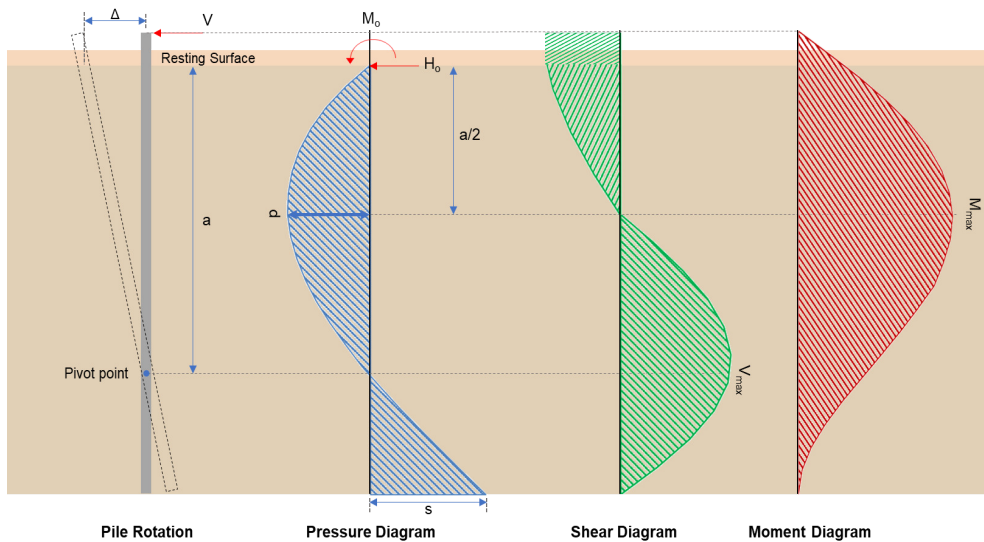
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.55911 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.64824$$

Status: **PASS**
Ratio: **0.820**

Status: **PASS**
Ratio: **0.650**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.101 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.033667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.181 \text{ kipft}) + ((-0.101 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.3937 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.3937 \text{ kipft/ft})}{(-0.033667 \text{ kip/ft})}$$

$$E = 71.099 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3937 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.033667 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.3937 \text{ kipft/ft})) + (4 \times (-0.033667 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8578 \text{ ft}$$

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

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

$$V_{max} = ((-0.033667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8578 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8578 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.2841 \text{ kip}$$

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

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

$$M_{max} = ((-0.033667 \text{ kip/ft}) \times (36 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(71.099 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 6.5132 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.929 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.30967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.285 \text{ kipft}) + ((0.929 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.76167 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.76167 \text{ kipft/ft})}{(0.30967 \text{ kip/ft})}$$

$$E = 2.4596 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.76167 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.30967 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.76167 \text{ kipft/ft})) + (4 \times (0.30967 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1252 \text{ ft}$$

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

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

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

$$V_{max} = ((0.30967 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1252 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1252 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 1.3236 \text{ kip}$$

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

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

$$M_{max} = ((0.30967 \text{ kip/ft}) \times (36 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.4596 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.33 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

Table 22.4.2.1

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

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.43 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.953 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.953 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.43 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.01071$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.43 \text{ kip} \rightarrow 13430 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13430 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.718 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.718 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.718 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.718 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.677 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.2841 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.2841 \text{ kip})}{(74.677 \text{ kip})}$$

$$Ratio = 0.030586$$

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

Considering z-direction:

$V_{max} = 1.3236 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(1.3236 \text{ kip})}{(74.677 \text{ kip})}$$

$$Ratio = 0.017725$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

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

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 6.5132 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.5132 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.10501$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$M_{max} = 3.33 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

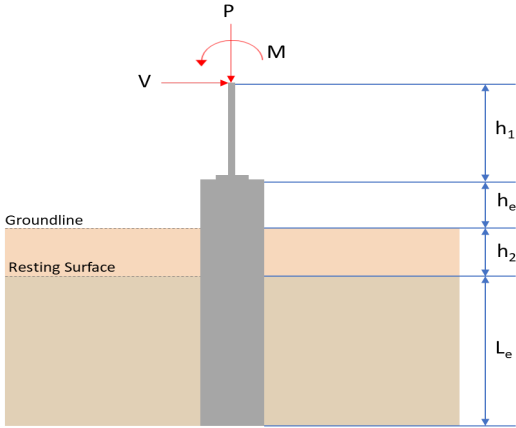
$$Ratio = \frac{M_{max}}{\phi M_n}$$

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

$$Ratio = \frac{(3.33 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.053686$$

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

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</p> <p>Pile shape: round $D = 36$ in - Pile diameter $L = 5.75$ 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="368 1061 1227 1162"> <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="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.124</td> <td>13.430</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.056</td> <td>-0.101</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.552</td> <td>-0.929</td> </tr> <tr> <td>M_x (kipft)</td> <td>-1.355</td> <td>-2.285</td> </tr> <tr> <td>M_z (kipft)</td> <td>4.129</td> <td>7.181</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	8.124	13.430	V_x (kip)	-0.056	-0.101	V_z (kip)	-0.552	-0.929	M_x (kipft)	-1.355	-2.285	M_z (kipft)	4.129	7.181	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.056 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.018667 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(4.129 \text{ kipft}) + ((-0.056 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.3763 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.4663 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.552 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.184 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.355 \text{ kipft}) + ((-0.552 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.45167 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.4375 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4663 \text{ ft}), (2.4375 \text{ ft})]$$

$$L_{e,req} = 5.466 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.466 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95061$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(8.124 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.1493 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.1493 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.57466$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 1.9167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.018667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.3763 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.3763 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (1.3763 \text{ kipft/ft})) + (4 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.857 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.3763 \text{ kipft/ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (1.3763 \text{ kipft/ft})) + (2 \times (-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24455 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (1.3763 \text{ kipft/ft})) + ((-0.018667 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.7541 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.857 \text{ ft})}{2}$$

$$p_a = 0.28928 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24455 \text{ kip/ft}^2)}{(0.28928 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84539$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.7541 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87431$$

Status: **PASS**
Ratio: **0.850**

Status: **PASS**
Ratio: **0.870**

Considering z-direction:

$H_o = -0.184 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.45167 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.45167 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.184 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.45167 \text{ kipft/ft})) + (4 \times (-0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1254 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.45167 \text{ kipft/ft})) + (3 \times (-0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.45167 \text{ kipft/ft})) + (2 \times (-0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = -0.087533 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.45167 \text{ kipft/ft})) + ((-0.184 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = -0.04409 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1254 \text{ ft})}{2}$$

$$p_a = 0.30941 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.087533 \text{ kip/ft}^2)}{(0.30941 \text{ kip/ft}^2)}$$

(0.0051 kip/ft)

$$Ratio = -0.28291$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

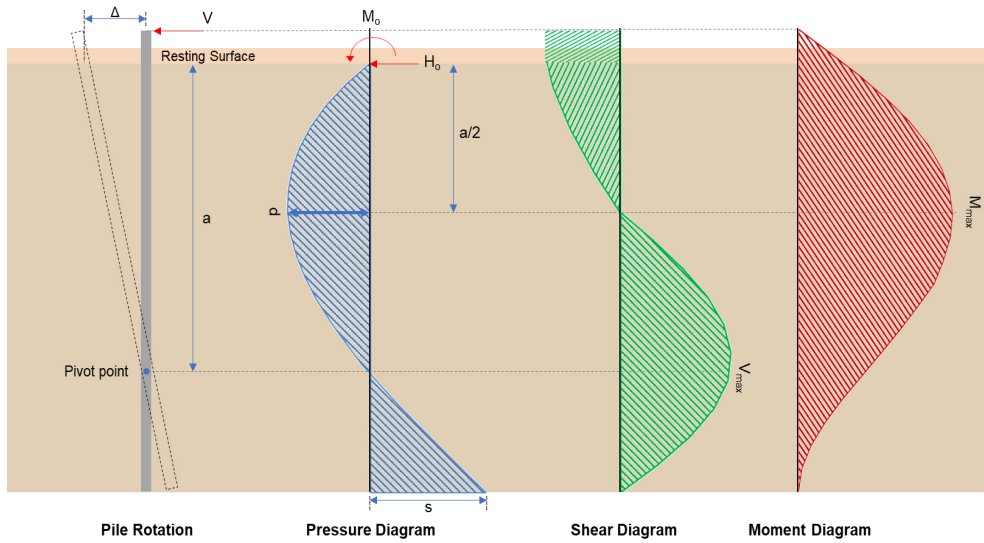
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.04409 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.051119$$

Status: **PASS**
Ratio: **-0.280**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.101 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.033667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.181 \text{ kipft}) + ((-0.101 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.3937 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.3937 \text{ kipft/ft})}{(-0.033667 \text{ kip/ft})}$$

$$E = 71.099 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3937 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.033667 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.3937 \text{ kipft/ft})) + (4 \times (-0.033667 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8578 \text{ ft}$$

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

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

$$V_{max} = ((-0.033667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8578 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8578 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.2841 \text{ kip}$$

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

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

$$M_{max} = ((-0.033667 \text{ kip/ft}) \times (36 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(71.099 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (71.099 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.8578 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 6.5132 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.929 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.30967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.285 \text{ kipft}) + ((-0.929 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.76167 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.76167 \text{ kipft/ft})}{(-0.30967 \text{ kip/ft})}$$

$$E = 2.4596 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.76167 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.30967 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.76167 \text{ kipft/ft})) + (4 \times (-0.30967 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1252 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.30967 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1252 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1252 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 1.3236 \text{ kip}$$

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

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

$$M_{max} = ((-0.30967 \text{ kip/ft}) \times (36 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.4596 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4596 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1252 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.33 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

Table 22.4.2.1

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

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

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.43 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.953 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.953 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10Ø: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.43 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.01071$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.43 \text{ kip} \rightarrow 13430 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13430 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.718 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.718 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.718 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.718 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.677 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.2841 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.2841 \text{ kip})}{(74.677 \text{ kip})}$$

$$Ratio = 0.030586$$

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

Considering z-direction:

$V_{max} = 1.3236 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(1.3236 \text{ kip})}{(74.677 \text{ kip})}$$

$$Ratio = 0.017725$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

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

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 6.5132 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.5132 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.10501$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$M_{max} = 3.33 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(3.33 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.053686$$

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

REFERENCES	CALCULATIONS	RESULTS
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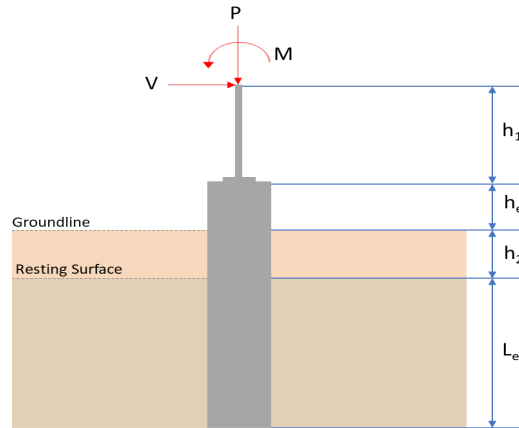
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	12.165	20.217
V_x (kip)	-0.086	-0.150
V_z (kip)	-0.091	-0.154
M_x (kipft)	-0.226	-0.385
M_z (kipft)	5.172	8.952

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.086 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.028667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.172 \text{ kipft}) + ((-0.086 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.724 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.856 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.091 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.030333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.226 \text{ kipft}) + ((-0.091 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.075333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6731 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[5.856 \text{ ft}, (1.6731 \text{ ft})]$$

$$L_{e,req} = 5.856 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.856 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.976$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(12.165 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.721 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.721 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.8605$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.028667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.724 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.724 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (1.724 \text{ kipft/ft})) + (4 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0312 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.724 \text{ kipft/ft})) + (3 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (1.724 \text{ kipft/ft})) + (2 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.27588 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (1.724 \text{ kipft/ft})) + ((-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.85767 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0312 \text{ ft})}{2}$$

$$p_a = 0.30234 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27588 \text{ kip/ft}^2)}{(0.30234 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91248$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.85767 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95297$$

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

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

Considering z-direction:

$H_o = -0.030333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.075333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075333 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.030333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.075333 \text{ kipft/ft})) + (4 \times (-0.030333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3085 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.075333 \text{ kipft/ft})) + (3 \times (-0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.075333 \text{ kipft/ft})) + (2 \times (-0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.014194 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.075333 \text{ kipft/ft})) + ((-0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = -0.0082032 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3085 \text{ ft})}{2}$$

$$p_a = 0.32314 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.014194 \text{ kip/ft}^2)}{(0.32314 \text{ kip/ft}^2)}$$

$$(0.0017 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.043927$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

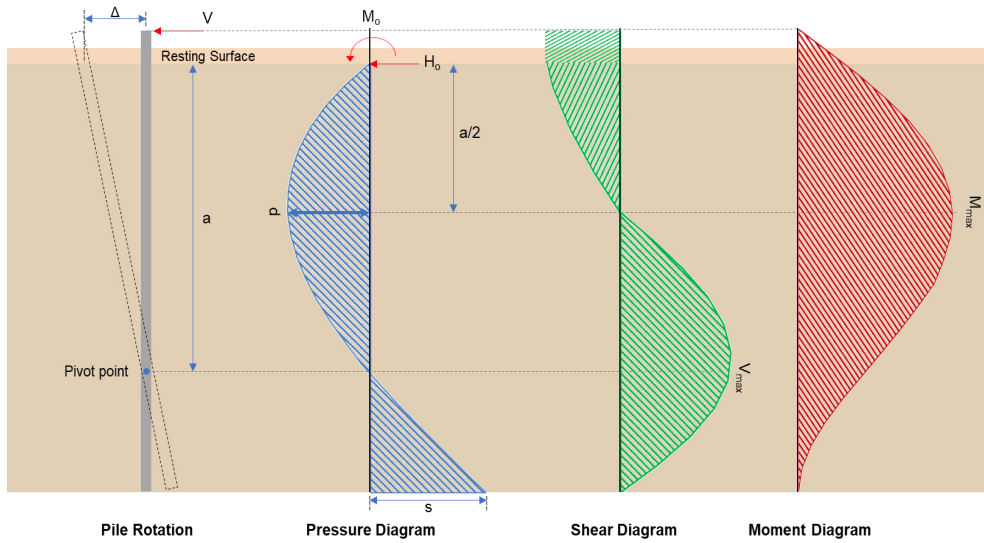
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0082032 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0091147$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.15 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.05 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(8.952 \text{ kipft}) + ((-0.15 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.984 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.984 \text{ kipft/ft})}{(-0.05 \text{ kip/ft})}$$

$$E = 59.68 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.984 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.984 \text{ kipft/ft})) + (4 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0314 \text{ ft}$$

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

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

$$V_{max} = ((-0.05 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0314 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0314 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.7474 \text{ kip}$$

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

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

$$M_{max} = ((-0.05 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(59.68 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.1593 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.154 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.051333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.385 \text{ kipft}) + ((-0.154 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.12833 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.12833 \text{ kipft/ft})}{(-0.051333 \text{ kip/ft})}$$

$$E = 2.5 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12833 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.051333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.12833 \text{ kipft/ft})) + (4 \times (-0.051333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3077 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.051333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3077 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3077 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21644 \text{ kip}$$

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

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

$$M_{max} = ((-0.051333 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.5 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.56709 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(20.217 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.741 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.741 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(20.217 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.016123$</p>	<p>Status: PASS Ratio: 0.020</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 20.217 \text{ kip} \rightarrow 20217 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(20217 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.87 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (77.87 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.87 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.87 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 75.426 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.7474 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(2.7474 \text{ kip})}{(75.426 \text{ kip})}$$

$$Ratio = 0.036425$$

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

Considering z-direction:

$V_{max} = 0.21644 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.21644 \text{ kip})}{(75.426 \text{ kip})}$$

$$Ratio = 0.0028695$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 8.1593 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.1593 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.13154$$

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

Considering z-direction:

$M_{max} = 0.56709 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.56709 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0091427$$

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

REFERENCES	CALCULATIONS	RESULTS
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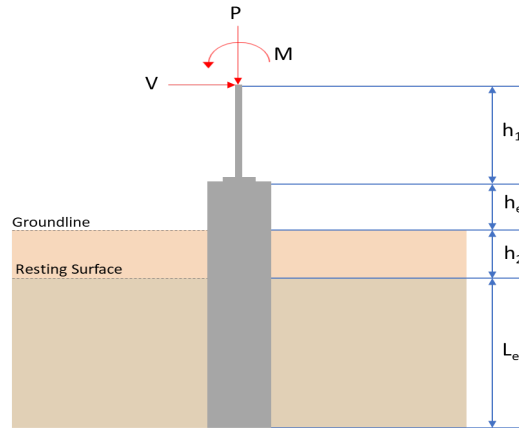
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	12.165	20.217
V_x (kip)	-0.086	-0.150
V_z (kip)	0.091	0.154
M_x (kipft)	0.226	0.385
M_z (kipft)	5.172	8.952

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.086 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.028667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.172 \text{ kipft}) + ((-0.086 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.724 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.856 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.091 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.030333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.226 \text{ kipft}) + ((0.091 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.075333 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.5605 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[5.856 \text{ ft}, (2.5605 \text{ ft})]$$

$$L_{e,req} = 5.856 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.856 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.976$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(12.165 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.721 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.721 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.8605$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.028667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.724 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.724 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (1.724 \text{ kipft/ft})) + (4 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0312 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.724 \text{ kipft/ft})) + (3 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (1.724 \text{ kipft/ft})) + (2 \times (-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.27588 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (1.724 \text{ kipft/ft})) + ((-0.028667 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.85767 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0312 \text{ ft})}{2}$$

$$p_a = 0.30234 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27588 \text{ kip/ft}^2)}{(0.30234 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91248$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.85767 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95297$$

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

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

Considering z-direction:

$H_o = 0.030333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.075333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075333 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.030333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.075333 \text{ kipft/ft})) + (4 \times (0.030333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3085 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.075333 \text{ kipft/ft})) + (3 \times (0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.075333 \text{ kipft/ft})) + (2 \times (0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.03982 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.075333 \text{ kipft/ft})) + ((0.030333 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.087094 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3085 \text{ ft})}{2}$$

$$p_a = 0.32314 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.03982 \text{ kip/ft}^2)}{(0.32314 \text{ kip/ft}^2)}$$

(0.087094 kip/ft²)

$$\text{Ratio} = 0.12323$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

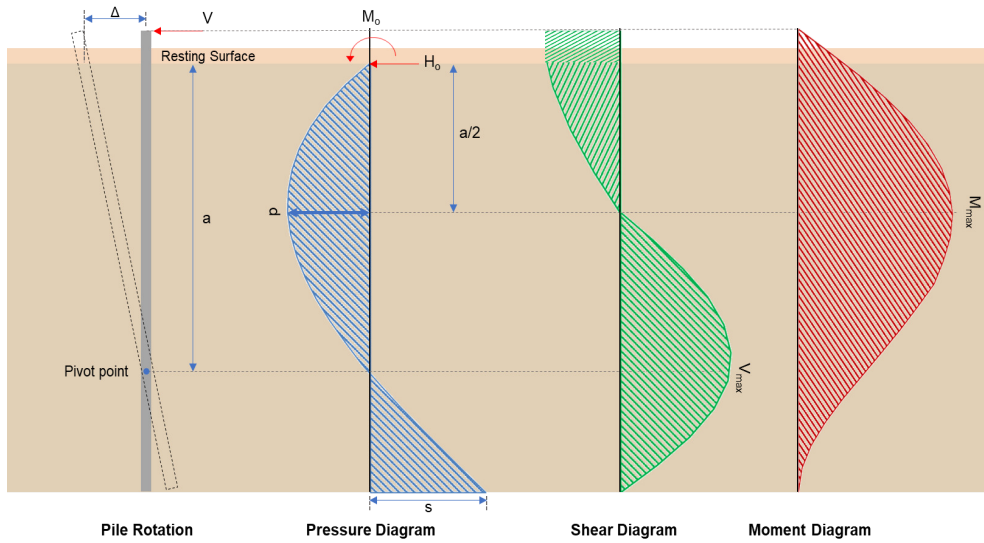
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.087094 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.096771$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.15 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.05 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(8.952 \text{ kipft}) + ((-0.15 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.984 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.984 \text{ kipft/ft})}{(-0.05 \text{ kip/ft})}$$

$$E = 59.68 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.984 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.984 \text{ kipft/ft})) + (4 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0314 \text{ ft}$$

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

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

$$V_{max} = ((-0.05 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0314 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0314 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.7474 \text{ kip}$$

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

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

$$M_{max} = ((-0.05 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(59.68 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (59.68 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0314 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.1593 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.154 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.051333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.385 \text{ kipft}) + ((0.154 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.12833 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.12833 \text{ kipft/ft})}{(0.051333 \text{ kip/ft})}$$

$$E = 2.5 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12833 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.051333 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.12833 \text{ kipft/ft})) + (4 \times (0.051333 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3077 \text{ ft}$$

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

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

$$\left[\frac{L_e}{2} \quad \text{or} \quad \frac{L_e}{3} \right]$$

$$V_{max} = ((0.051333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3077 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3077 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21644 \text{ kip}$$

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

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

$$M_{max} = ((0.051333 \text{ kip/ft}) \times (36 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.5 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3077 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.56709 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(20.217 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.741 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.741 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(20.217 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.016123$</p>	<p>Status: PASS Ratio: 0.020</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 20.217 \text{ kip} \rightarrow 20217 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(20217 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.87 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (77.87 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.87 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.87 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 75.426 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.7474 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(2.7474 \text{ kip})}{(75.426 \text{ kip})}$$

$$Ratio = 0.036425$$

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

Considering z-direction:

$V_{max} = 0.21644 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.21644 \text{ kip})}{(75.426 \text{ kip})}$$

$$Ratio = 0.0028695$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 8.1593 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.1593 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.13154$$

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

Considering z-direction:

$M_{max} = 0.56709 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.56709 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0091427$$

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