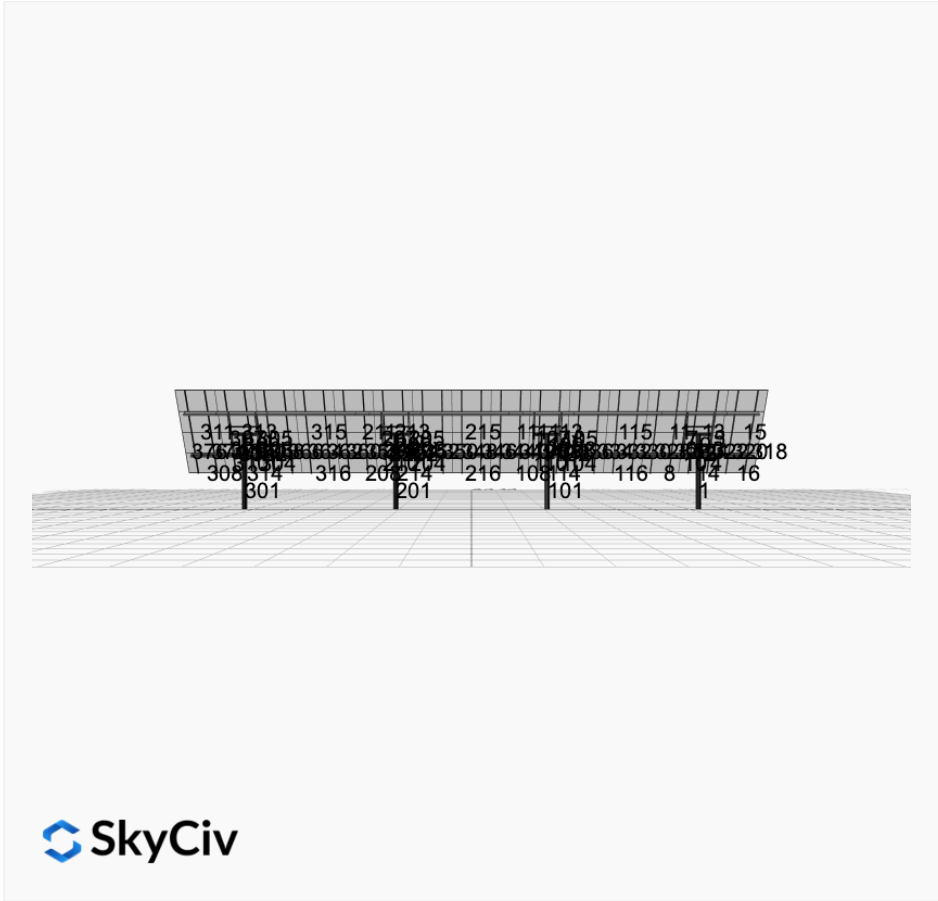


Project Name: YNP - Lamar Buffalo Ranch - 8in - V1Jb
Date: Wed Jan 22 2025
Number of Modules: 60
Location: VQW7+6QG, Fossil Forest, WY 82190, USA
Number of Poles: 4
Unique ID: 4P-22.5-8TOP-HD-57-L-4Hx15W-CK74
Date Sold:
Dealer: _____



Array Dimensions N/S	14.83 ft
Array Dimensions E/W	86.25 ft
Winter Tilt Angle	55
Front Edge Clearance	5 ft

MT Solar Bill of Materials (4P-22.5-8TOP-HD-57-L-4Hx15W-CK74)

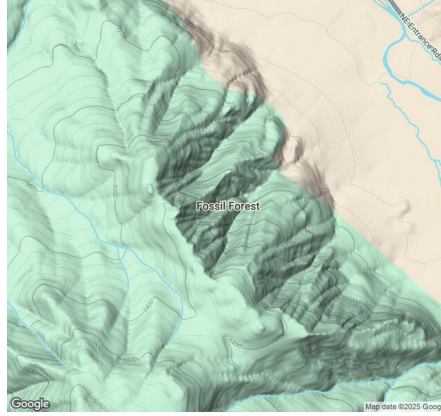
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	12
MTS-CLAMP-HOOK-4PK	Hook Clamp	15

Rail Bill of Materials

Part	Qty
Rails (176in)	30
Rail Attachment	60
Module Mid Clamp	90

Part	Qty
Module End Clamp	60
Ground Lug	15

Site Details:



Site Address: VQW7+6QG, Fossil Forest, WY 82190, USA

Array Specification

Duty Classification:	HD
Module Width:	44.00 in
Module Length:	68.00in
Number of Rows:	4
Number of Columns:	15
Total Number of Modules:	60
Winter Tilt Angle:	55
Front Edge Clearance:	5
Total Array Height at Tilt:	17.15 ft
Total Frame Length:	84.50 ft
Frame Weight:	5069 lbs
Array Dimensions N/S:	14.83 ft
Array Dimensions E/W:	86.25 ft
Rail Length:	178.00 in
Rail Spacing:	2.88 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.08 ft
Number of Poles:	4
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.00 ft
Foundation Volume:	22.685 y ³

Site Info

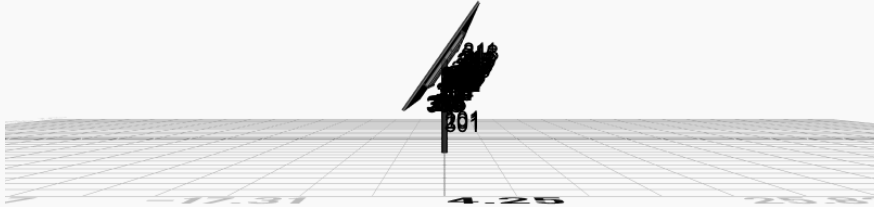
Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	VQW7+6QG, Fossil Forest, WY 82190, USA
Wind Speed:	101 mph

Snow Load:

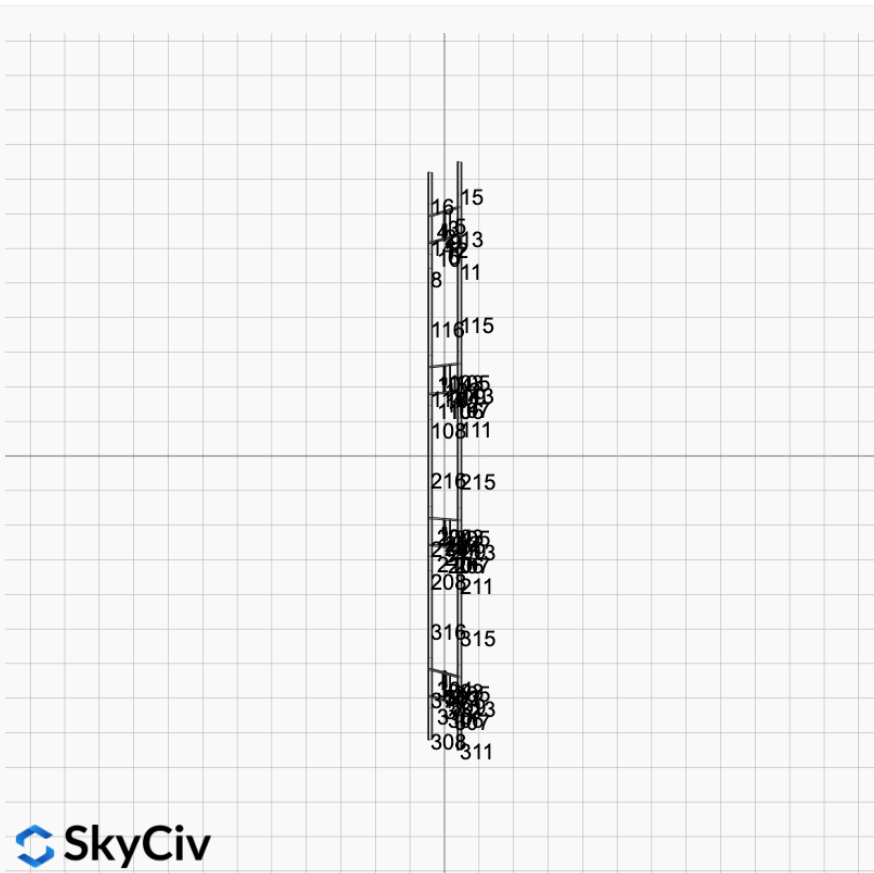
130 psf

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.



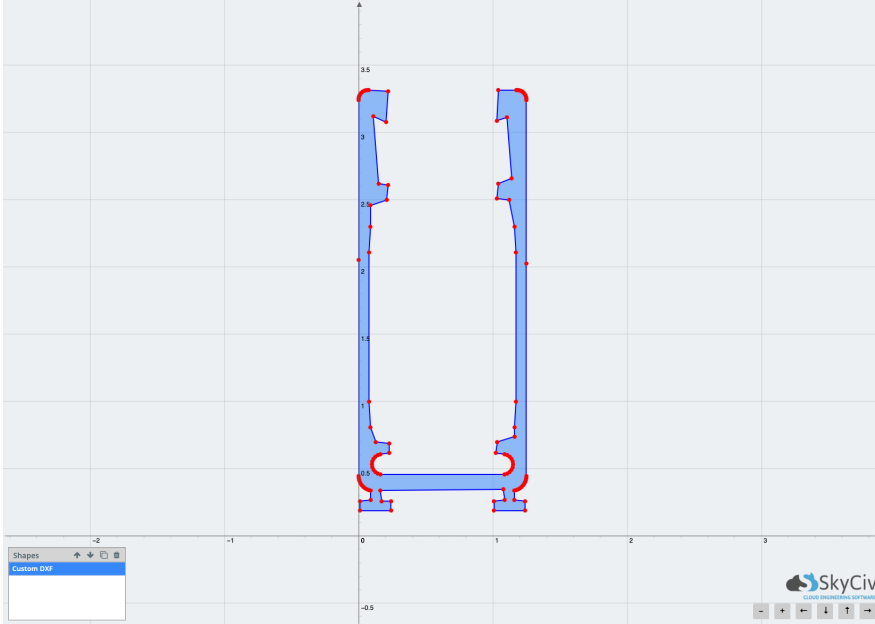
 SkyCiv



 SkyCiv

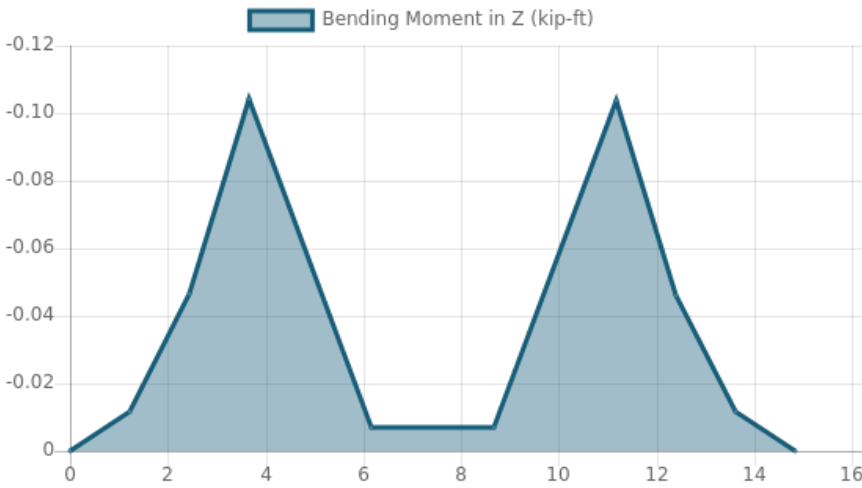
Rail Design Check

Rail Length: 14.833333333333334 ft
Additional Restraints Required: None
Tributary Width: 2.875 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0354 kip/ft
Snow (Y): -0.0505 kip/ft
Wind uplift Case A: 0.0499 kip/ft
Wind downforce Case A: 0.0499 kip/ft
Dead (Panel load) (X): 0.0078 kip/ft
Dead (Panel load) (Y): -0.0111 kip/ft

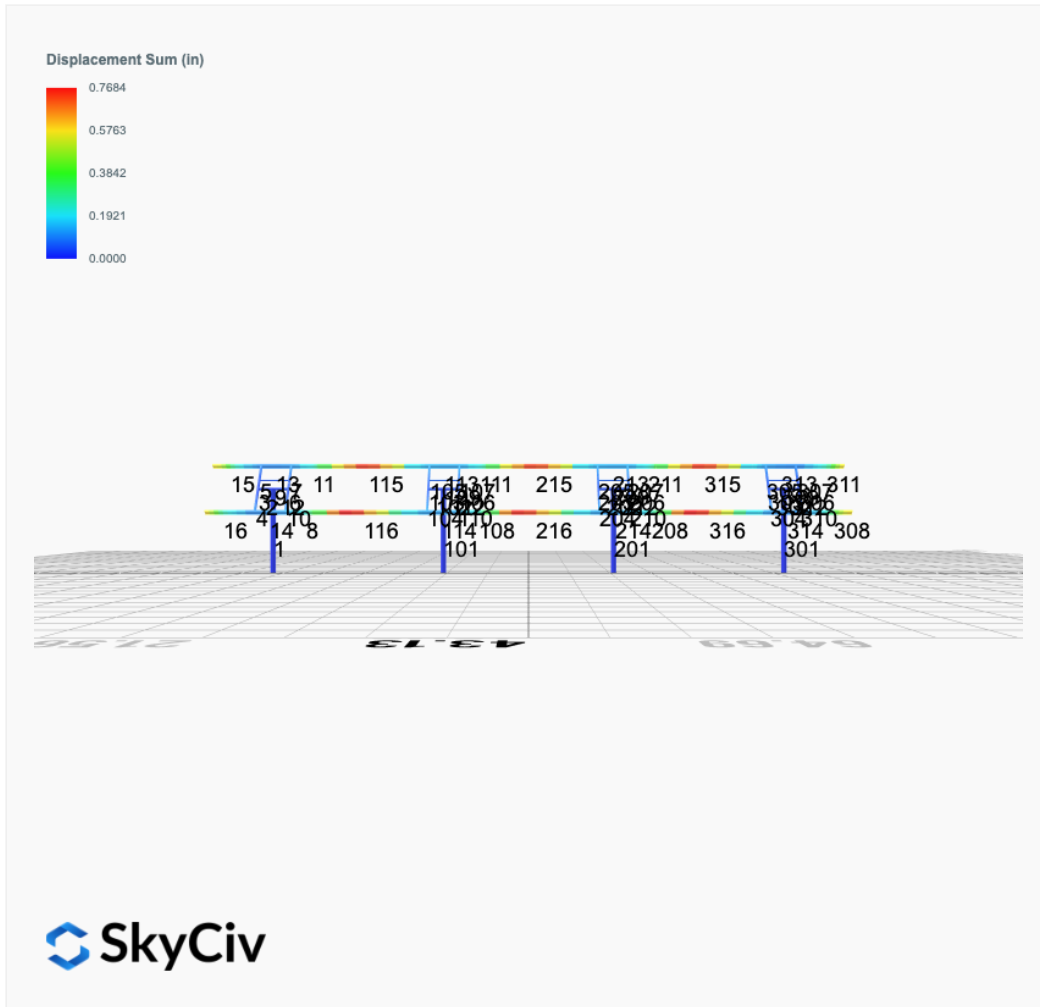


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	17.56272071	0.509	PASS
Material Yield	34.5	17.56272071	0.509	PASS
Material Strength	37	17.56272071	0.475	PASS

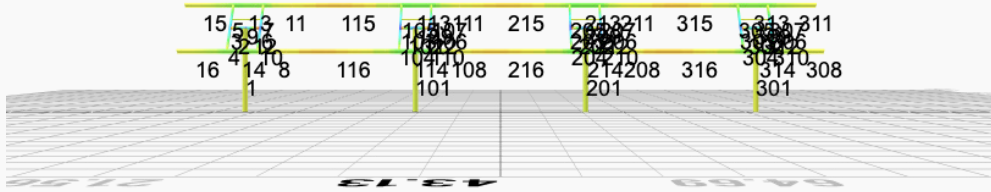
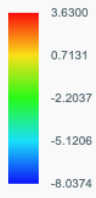
Member 1, ULS: 1. 1.4D



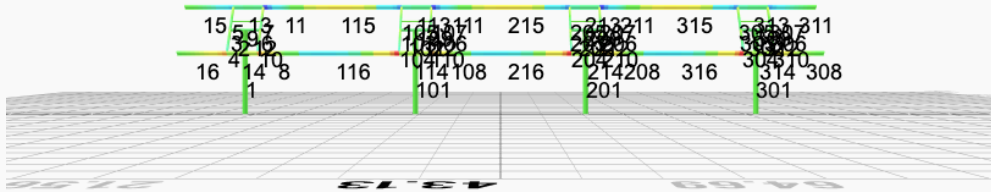
FEM Results (Envelope Worst Case for each member)



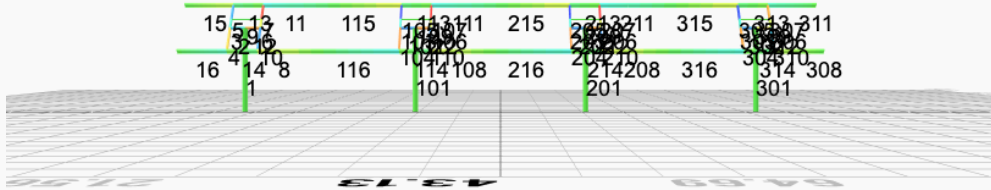
Top Bending Stress Z (ksi)



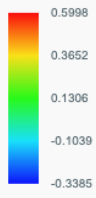
Top Bending Stress Y (ksi)



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	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 2. D + L	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 3. D + (S or Lr or R)	0.0267	5.9153	0.1088	0.3647	-0.0637	-0.2510
ULS: 3. D + (S or Lr or R)	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0223	5.0179	0.0906	0.3037	-0.0530	-0.2072
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 5b. D + 0.7E	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0223	5.0179	0.0906	0.3037	-0.0530	-0.2072
ULS: 8. 0.6D + 0.7E	0.0053	1.3955	0.0217	0.0725	-0.0126	-0.0455
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5386	4.0995	0.1082	0.3395	-0.4517	28.4592
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5554	0.5525	-0.0346	-0.0939	0.4029	-28.1269
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8884	6.3482	0.1446	0.4677	-0.3760	21.1941
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0223	5.0179	0.0906	0.3037	-0.0530	-0.2072
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9322	3.6879	0.0375	0.1427	0.2649	-21.2455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0223	5.0179	0.0906	0.3037	-0.0530	-0.2072
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9017	3.6561	0.0902	0.2848	-0.3440	21.3255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9188	0.9958	-0.0170	-0.0402	0.2969	-21.1142
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0089	2.3258	0.0361	0.1208	-0.0209	-0.0759
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5422	3.1692	0.0937	0.2912	-0.4433	28.4896
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0053	1.3955	0.0217	0.0725	-0.0126	-0.0455
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5519	-0.3778	-0.0491	-0.1422	0.4112	-28.0966
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0053	1.3955	0.0217	0.0725	-0.0126	-0.0455

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	10.0127
Shear X	-4.2649
Shear Z	0.2219
Moment X	0.7281
Moment Y (Twist)	0.7715
Moment Z	48.0814

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	6.3482
Shear X	-2.5554
Shear Z	0.1446
Moment X	0.4677
Moment Y (Twist)	0.4517
Moment Z	28.4896

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	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 2. D + L	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 3. D + (S or Lr or R)	-0.0267	6.7092	-0.0035	-0.0117	0.0190	0.3268
ULS: 3. D + (S or Lr or R)	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0223	5.6796	-0.0029	-0.0098	0.0158	0.2738

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 5b. D + 0.7E	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0223	5.6796	-0.0029	-0.0098	0.0158	0.2738
ULS: 8. 0.6D + 0.7E	-0.0053	1.5544	-0.0007	-0.0024	0.0038	0.0689
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9224	4.6408	0.0096	0.0276	-0.0709	32.6109
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9056	0.5401	-0.0112	-0.0333	0.0792	-31.7806
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2074	7.2172	0.0051	0.0139	-0.0421	24.6459
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0223	5.6796	-0.0029	-0.0098	0.0158	0.2738
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1636	4.1417	-0.0104	-0.0318	0.0704	-23.6478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0223	5.6796	-0.0029	-0.0098	0.0158	0.2738
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1941	4.1283	0.0069	0.0197	-0.0516	24.4869
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1770	1.0528	-0.0087	-0.0260	0.0610	-23.8068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0089	2.5907	-0.0012	-0.0039	0.0063	0.1148
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9189	3.6046	0.0100	0.0292	-0.0735	32.5650
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0053	1.5544	-0.0007	-0.0024	0.0038	0.0689
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9092	-0.4961	-0.0107	-0.0317	0.0766	-31.8265
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0053	1.5544	-0.0007	-0.0024	0.0038	0.0689

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.4065
Shear X	-4.8742
Shear Z	-0.0201
Moment X	-0.0598
Moment Y (Twist)	0.1408
Moment Z	55.2688

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.2172
Shear X	-2.9224
Shear Z	-0.0112
Moment X	-0.0333
Moment Y (Twist)	0.0792
Moment Z	32.6109

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	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 2. D + L	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 3. D + (S or Lr or R)	-0.0267	6.7092	0.0035	0.0116	-0.0188	0.3268
ULS: 3. D + (S or Lr or R)	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0223	5.6796	0.0029	0.0097	-0.0157	0.2738
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 5b. D + 0.7E	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0223	5.6796	0.0029	0.0097	-0.0157	0.2738
ULS: 8. 0.6D + 0.7E	-0.0053	1.5544	0.0007	0.0023	-0.0038	0.0689
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9224	4.6408	-0.0096	-0.0276	0.0710	32.6109
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9056	0.5401	0.0112	0.0333	-0.0791	-31.7806
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2074	7.2172	-0.0051	-0.0140	0.0423	24.6459
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0223	5.6796	0.0029	0.0097	-0.0157	0.2738
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1636	4.1417	0.0104	0.0317	-0.0703	-23.6478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0223	5.6796	0.0029	0.0097	-0.0157	0.2738
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1941	4.1283	-0.0069	-0.0198	0.0516	24.4869
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1770	1.0528	0.0087	0.0259	-0.0609	-23.8068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0089	2.5907	0.0012	0.0039	-0.0063	0.1148
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9189	3.6046	-0.0100	-0.0292	0.0735	32.5650
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0053	1.5544	0.0007	0.0023	-0.0038	0.0689
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9092	-0.4961	0.0107	0.0317	-0.0766	-31.8265
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0053	1.5544	0.0007	0.0023	-0.0038	0.0689

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.4065
Shear X	-4.8742
Shear Z	0.0201
Moment X	0.0602
Moment Y (Twist)	0.1407
Moment Z	55.2689

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.2172
Shear X	-2.9224
Shear Z	0.0112
Moment X	0.0333
Moment Y (Twist)	0.0791
Moment Z	32.6109

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	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 2. D + L	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 3. D + (S or Lr or R)	0.0267	5.9153	-0.1088	-0.3650	0.0638	-0.2509
ULS: 3. D + (S or Lr or R)	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0223	5.0179	-0.0906	-0.3040	0.0531	-0.2071
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 5b. D + 0.7E	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0223	5.0179	-0.0906	-0.3040	0.0531	-0.2071
ULS: 8. 0.6D + 0.7E	0.0053	1.3955	-0.0217	-0.0725	0.0126	-0.0455
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5386	4.0995	-0.1082	-0.3395	0.4517	28.4593
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5554	0.5525	0.0346	0.0938	-0.4029	-28.1269
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8884	6.3482	-0.1446	-0.4680	0.3761	21.1942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0223	5.0179	-0.0906	-0.3040	0.0531	-0.2071
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9322	3.6879	-0.0375	-0.1430	-0.2648	-21.2454
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0223	5.0179	-0.0906	-0.3040	0.0531	-0.2071
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9017	3.6561	-0.0902	-0.2849	0.3440	21.3255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9188	0.9958	0.0170	0.0401	-0.2969	-21.1141
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0089	2.3258	-0.0361	-0.1209	0.0209	-0.0758

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5422	3.1692	-0.0937	-0.2912	0.4433	28.4896
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0053	1.3955	-0.0217	-0.0725	0.0126	-0.0455
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5519	-0.3778	0.0491	0.1422	-0.4112	-28.0966
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0053	1.3955	-0.0217	-0.0725	0.0126	-0.0455

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.0126
Shear X	-4.2649
Shear Z	-0.2219
Moment X	-0.7300
Moment Y (Twist)	0.7719
Moment Z	48.0822

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.3482
Shear X	-2.5554
Shear Z	-0.1446
Moment X	-0.4680
Moment Y (Twist)	0.4517
Moment Z	28.4896

Project Details

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

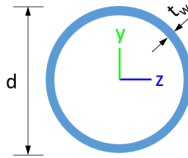


Design Input Information

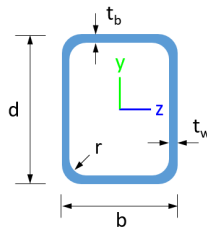
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

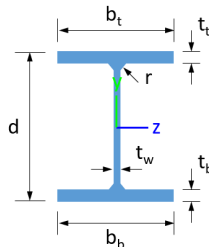
Section Dimensions



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



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.03,1.03,1.04,1.03,1.0	300	200	1
114	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.04,1.03,1.04,1.03,1.04,1.03,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
115	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.17,1.13,1.17,1.15,1.17,1.13,1.17,1.16,1.17,1.21,1.17,1.15,1.17,1.1	300	200	1
116	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.17,1.17,1.17,1.16,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.1	300	200	1
201	9	23.26	23.26	11.08	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.1	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.1	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1
208	19	1.33	1.33	2.05	2.30,2.30,2.30,2.29,2.30,2.30,2.32,2.30,2.35,2.30,2.32,2.30,2.33,2.30,2.31,2.29,2.26,2.29,2.32,2.30,2.3	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1
211	19	1.33	1.33	2.05	2.31,2.31,2.31,2.31,2.31,2.31,2.14,2.31,2.05,2.31,2.13,2.31,2.10,2.31,2.33,2.31,2.05,2.31,2.14,2.31,1.9	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.03,1.03,1.04,1.03,1.0	300	200	1
214	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.04,1.03,1.04,1.03,1.04,1.03,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
215	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.17,1.14,1.17,1.16,1.17,1.15,1.17,1.16,1.17,1.21,1.17,1.16,1.17,1.1	300	200	1
216	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.1	300	200	1
301	9	23.26	23.26	11.08	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.1	300	200	1
304	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1
305	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1
306	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
307	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1
308	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.69,1.6	300	200	1
311	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
312	5	1.30	1.30	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.06,1.08,1.07,1.08,1.06,1.08,1.07,1.08,1.10,1.08,1.07,1.08,1.0	300	200	1
314	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.07,1.08,1.07,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.0	300	200	1

212	196.55	190.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.55	6.12	40.24	43.62
214	133.20	85.85	23.57	6.12	40.24	43.62
215	133.20	46.28	12.35	6.12	40.24	43.62
216	133.20	46.28	12.61	6.12	40.24	43.62
301	377.97	195.36	83.29	83.29	113.39	113.39
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	32.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	32.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.20	6.12	40.24	43.62
314	133.20	85.85	24.54	6.12	40.24	43.62
315	133.20	46.28	12.32	6.12	40.24	43.62
316	133.20	46.28	12.25	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.051	0.577	0.021	0.038	0.002	0.604	#13	0.475	Not Required	Pass
2	0.007	0.277	0.182	0.071	0.034	0.408	#21	0.035	Not Required	Pass
3	0.013	0.414	0.068	0.041	0.009	0.447	#21	0.045	Not Required	Pass
4	0.012	0.413	0.246	0.041	0.051	0.559	#21	0.080	Not Required	Pass
5	0.012	0.257	0.247	0.041	0.063	0.316	#21	0.074	Not Required	Pass
6	0.016	0.489	0.102	0.049	0.012	0.574	#21	0.045	Not Required	Pass
7	0.017	0.303	0.331	0.048	0.084	0.377	#21	0.074	Not Required	Pass
8	0.003	0.046	0.344	0.034	0.028	0.363	#21	0.095	Not Required	Pass
9	0.028	0.033	0.072	0.003	0.002	0.091	#13	0.204	Not Required	Pass
10	0.017	0.474	0.314	0.047	0.065	0.646	#21	0.080	Not Required	Pass
11	0.004	0.041	0.355	0.035	0.028	0.381	#21	0.095	Not Required	Pass
12	0.006	0.356	0.217	0.091	0.040	0.507	#13	0.035	Not Required	Pass
13	0.011	0.181	0.751	0.043	0.035	0.893	#21	0.286	Not Required	Pass
14	0.014	0.177	0.738	0.042	0.035	0.869	#21	0.190	Not Required	Pass
15	0.000	0.065	0.303	0.022	0.018	0.365	#21	Not Required	Not Required	Pass
16	0.000	0.065	0.303	0.022	0.018	0.365	#21	Not Required	Not Required	Pass
101	0.058	0.664	0.002	0.043	0.000	0.686	#13	0.475	Not Required	Pass
102	0.007	0.362	0.226	0.093	0.040	0.525	#21	0.035	Not Required	Pass
103	0.016	0.509	0.082	0.051	0.002	0.583	#21	0.045	Not Required	Pass
104	0.016	0.515	0.316	0.051	0.065	0.700	#21	0.080	Not Required	Pass
105	0.016	0.316	0.333	0.050	0.085	0.397	#21	0.074	Not Required	Pass
106	0.016	0.521	0.082	0.052	0.002	0.587	#21	0.045	Not Required	Pass
107	0.016	0.323	0.330	0.052	0.084	0.401	#21	0.074	Not Required	Pass
108	0.003	0.050	0.342	0.035	0.028	0.392	#21	0.095	Not Required	Pass
109	0.033	0.028	0.056	0.001	0.000	0.089	#21	0.204	Not Required	Pass
110	0.016	0.518	0.313	0.052	0.065	0.697	#21	0.080	Not Required	Pass

111	0.004	0.045	0.352	0.035	0.028	0.398	#21	0.095	Not Required	Pass
112	0.007	0.364	0.232	0.093	0.042	0.529	#21	0.035	Not Required	Pass
113	0.011	0.187	0.754	0.044	0.035	0.924	#21	0.286	Not Required	Pass
114	0.015	0.204	0.743	0.045	0.035	0.923	#21	0.286	Not Required	Pass
115	0.010	0.331	0.408	0.035	0.028	0.724	#21	0.601	Not Required	Pass
116	0.003	0.324	0.405	0.036	0.028	0.713	#21	0.601	Not Required	Pass
201	0.058	0.664	0.002	0.043	0.000	0.686	#13	0.475	Not Required	Pass
202	0.007	0.363	0.232	0.093	0.042	0.529	#21	0.035	Not Required	Pass
203	0.016	0.521	0.082	0.052	0.002	0.587	#21	0.045	Not Required	Pass
204	0.016	0.518	0.313	0.052	0.065	0.697	#21	0.080	Not Required	Pass
205	0.016	0.323	0.330	0.052	0.084	0.401	#21	0.074	Not Required	Pass
206	0.016	0.509	0.082	0.051	0.002	0.583	#21	0.045	Not Required	Pass
207	0.016	0.316	0.333	0.050	0.085	0.397	#21	0.074	Not Required	Pass
208	0.003	0.047	0.348	0.036	0.028	0.395	#21	0.095	Not Required	Pass
209	0.033	0.028	0.056	0.001	0.000	0.089	#21	0.204	Not Required	Pass
210	0.016	0.515	0.316	0.051	0.065	0.700	#21	0.080	Not Required	Pass
211	0.004	0.040	0.358	0.035	0.028	0.399	#21	0.095	Not Required	Pass
212	0.007	0.362	0.226	0.093	0.040	0.525	#21	0.035	Not Required	Pass
213	0.011	0.187	0.754	0.044	0.035	0.925	#21	0.286	Not Required	Pass
214	0.015	0.204	0.743	0.045	0.035	0.922	#21	0.286	Not Required	Pass
215	0.010	0.309	0.409	0.035	0.028	0.698	#21	0.601	Not Required	Pass
216	0.003	0.287	0.405	0.035	0.028	0.677	#21	0.601	Not Required	Pass
301	0.051	0.577	0.021	0.038	0.002	0.604	#13	0.475	Not Required	Pass
302	0.006	0.356	0.217	0.091	0.040	0.507	#13	0.035	Not Required	Pass
303	0.016	0.489	0.102	0.049	0.012	0.574	#21	0.045	Not Required	Pass
304	0.017	0.474	0.314	0.047	0.065	0.646	#21	0.080	Not Required	Pass
305	0.017	0.303	0.331	0.048	0.084	0.377	#21	0.074	Not Required	Pass
306	0.013	0.414	0.068	0.041	0.009	0.447	#21	0.045	Not Required	Pass
307	0.012	0.257	0.247	0.041	0.063	0.316	#21	0.074	Not Required	Pass
308	0.000	0.065	0.303	0.022	0.018	0.365	#21	Not Required	Not Required	Pass
309	0.028	0.033	0.072	0.003	0.002	0.091	#13	0.204	Not Required	Pass
310	0.012	0.413	0.246	0.041	0.051	0.559	#21	0.080	Not Required	Pass
311	0.000	0.065	0.303	0.022	0.018	0.365	#21	Not Required	Not Required	Pass
312	0.007	0.277	0.182	0.071	0.034	0.408	#21	0.035	Not Required	Pass
313	0.011	0.181	0.750	0.043	0.035	0.893	#21	0.190	Not Required	Pass
314	0.014	0.177	0.738	0.042	0.035	0.869	#21	0.286	Not Required	Pass
315	0.010	0.331	0.409	0.035	0.028	0.726	#21	0.601	Not Required	Pass
316	0.003	0.329	0.404	0.034	0.028	0.717	#21	0.601	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis

KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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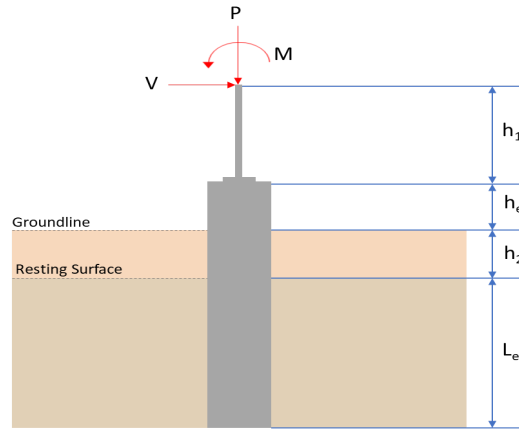
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 6$ 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)	6.348	10.013
V_x (kip)	-2.555	-4.265
V_z (kip)	0.145	0.222
M_x (kipft)	0.468	0.728
M_z (kipft)	28.490	48.081

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,

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

$$H_o = \frac{(-2.555 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.32548 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.49 \text{ kipft}) + ((-2.555 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 3.6293 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6469 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.145 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.018471 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.468 \text{ kipft}) + ((0.145 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.059618 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9017 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6469 \text{ ft}), (1.9017 \text{ ft})]$$

$$L_{e,req} = 5.647 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.647 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.94117$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.348 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.25392 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.25392 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12696$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.32548 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.6293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.6293 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.6293 \text{ kipft/ft})) + (4 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.132 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6293 \text{ kipft/ft})) + (3 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.6293 \text{ kipft/ft})) + (2 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.2237 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.6293 \text{ kipft/ft})) + ((-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.88429 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.132 \text{ ft})}{2}$$

$$p_a = 0.3099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.2237 \text{ kip/ft}^2)}{(0.3099 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.72184$$

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

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

$$\text{Ratio} = \frac{(0.88429 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98254$$

Status: **PASS**
Ratio: **0.720**

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

Considering z-direction:

$H_o = 0.018471 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.059618 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.059618 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.018471 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.059618 \text{ kipft/ft})) + (4 \times (0.018471 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2767 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.059618 \text{ kipft/ft})) + (3 \times (0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.059618 \text{ kipft/ft})) + (2 \times (0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.016957 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.059618 \text{ kipft/ft})) + ((0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.038344 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2767 \text{ ft})}{2}$$

$$p_a = 0.32075 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.016957 \text{ kip/ft}^2)}{(0.32075 \text{ kip/ft}^2)}$$

$$Ratio = 0.052866$$

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

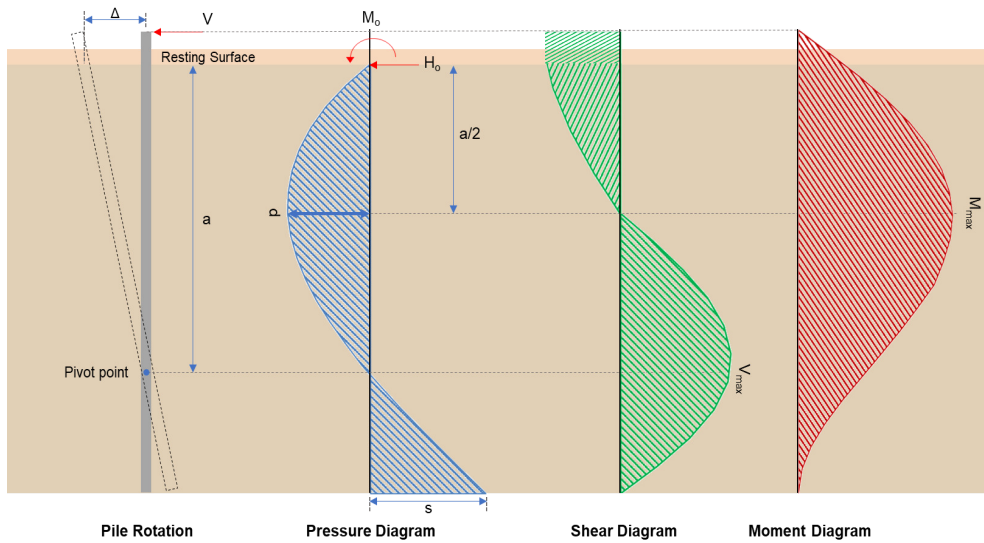
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.038344 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.042604$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.54331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(48.081 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 6.125 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.125 \text{ kipft/ft})}{(-0.54331 \text{ kip/ft})}$$

$$E = 11.273 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.125 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.125 \text{ kipft/ft})) + (4 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(6 \times (6.125 \text{ kipft/ft})) + (4 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft}))}{}$$

$$a = 4.1309 \text{ ft}$$

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

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

$$V_{max} = ((-0.54331 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.273 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1309 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.273 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1309 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.824 \text{ kip}$$

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

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

$$M_{max} = ((-0.54331 \text{ kip/ft}) \times (60 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(11.273 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.273 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.273 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 30.992 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.222 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.02828 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.728 \text{ kipft}) + ((0.222 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.092739 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.092739 \text{ kipft/ft})}{(0.02828 \text{ kip/ft})}$$

$$E = 3.2793 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.092739 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.02828 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.092739 \text{ kipft/ft})) + (4 \times (0.02828 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2748 \text{ ft}$$

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

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

$$V_{max} = ((0.02828 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2793 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2748 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2793 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2748 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23084 \text{ kip}$$

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

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

$$M_{max} = ((0.02828 \text{ kip/ft}) \times (60 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.2793 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2748 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2793 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2748 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2793 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2748 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.61674 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.013 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.85 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.85 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.013 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023948$</p>	<p>Status: PASS Ratio: 0.000</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>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.013 \text{ kip} \rightarrow 10013 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(10013 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.45 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.45 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.45 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.45 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.15 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.824 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.824 \text{ kip})}{(152.15 \text{ kip})}$$

$$Ratio = 0.071145$$

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

Considering z-direction:

$V_{max} = 0.23084 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.23084 \text{ kip})}{(152.15 \text{ kip})}$$

$$Ratio = 0.0015172$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 30.992 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(30.992 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.063573$$

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

Considering z-direction:

$M_{max} = 0.61674 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61674 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.0012651$$

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

REFERENCES	CALCULATIONS	RESULTS
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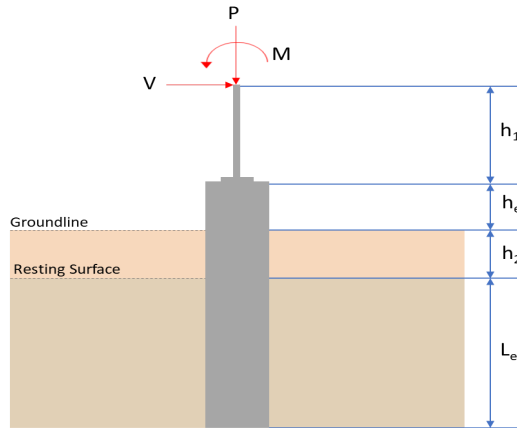
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 6$ 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)	6.348	10.013
V_x (kip)	-2.555	-4.265
V_z (kip)	-0.145	-0.222
M_x (kipft)	-0.468	-0.730
M_z (kipft)	28.490	48.082

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,

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

$$H_o = \frac{(-2.555 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.32548 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.49 \text{ kipft}) + ((-2.555 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 3.6293 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6469 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.145 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.018471 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.468 \text{ kipft}) + ((-0.145 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.059618 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4652 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6469 \text{ ft}), (1.4652 \text{ ft})]$$

$$L_{e,req} = 5.647 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.647 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.94117$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.348 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.25392 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.25392 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12696$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.32548 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.6293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.6293 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.6293 \text{ kipft/ft})) + (4 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.132 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6293 \text{ kipft/ft})) + (3 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.6293 \text{ kipft/ft})) + (2 \times (-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.2237 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.6293 \text{ kipft/ft})) + ((-0.32548 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.88429 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.132 \text{ ft})}{2}$$

$$p_a = 0.3099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2237 \text{ kip/ft}^2)}{(0.3099 \text{ kip/ft}^2)}$$

$$Ratio = 0.72184$$

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

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

$$Ratio = \frac{(0.88429 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.98254$$

Status: **PASS**
Ratio: **0.720**

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

Considering z-direction:

$H_o = -0.018471 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.059618 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.059618 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.018471 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.059618 \text{ kipft/ft})) + (4 \times (-0.018471 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2767 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.059618 \text{ kipft/ft})) + (3 \times (-0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 [(3 \times (0.059618 \text{ kipft/ft})) + (2 \times (-0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.0043019 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.059618 \text{ kipft/ft})) + ((-0.018471 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.0014013 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2767 \text{ ft})}{2}$$

$$p_a = 0.32075 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0043019 \text{ kip/ft}^2)}{(0.32075 \text{ kip/ft}^2)}$$

$$Ratio = -0.013412$$

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

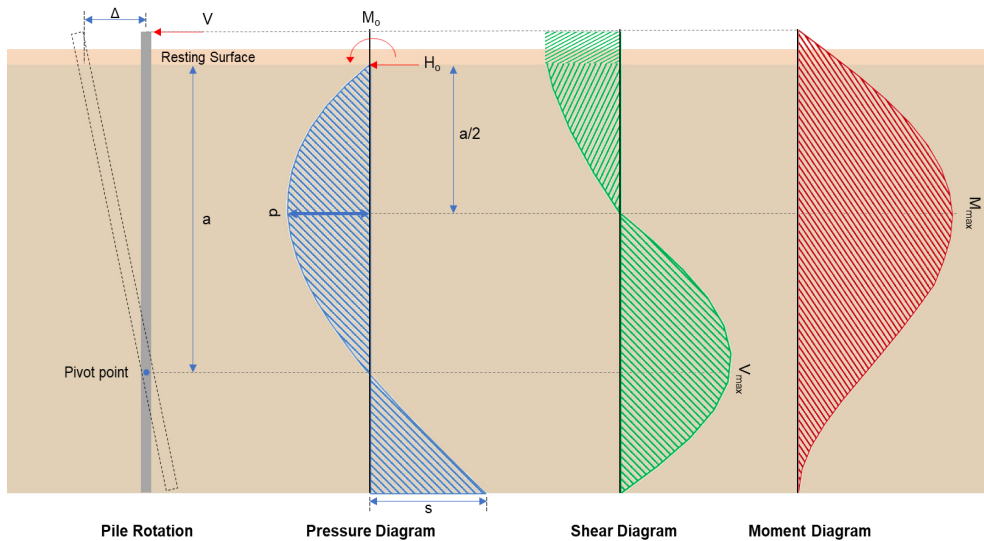
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0014013 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.001557$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.54331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(48.082 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 6.1251 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.1251 \text{ kipft/ft})}{(-0.54331 \text{ kip/ft})}$$

$$E = 11.274 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1251 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.1251 \text{ kipft/ft})) + (4 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.54331 \text{ kip/ft}) \times (60 \text{ in})}{(6 \times (6.1251 \text{ kipft/ft})) + (4 \times (-0.54331 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1309 \text{ ft}$$

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

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

$$V_{max} = ((-0.54331 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.274 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1309 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.274 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1309 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.825 \text{ kip}$$

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

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

$$M_{max} = ((-0.54331 \text{ kip/ft}) \times (60 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(11.274 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.274 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.274 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1309 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 30.992 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.222 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.02828 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.73 \text{ kipft}) + ((-0.222 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.092994 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.092994 \text{ kipft/ft})}{(-0.02828 \text{ kip/ft})}$$

$$E = 3.2883 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.092994 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.02828 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.092994 \text{ kipft/ft})) + (4 \times (-0.02828 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2744 \text{ ft}$$

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

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

$$V_{max} = ((-0.02828 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2883 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2744 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2883 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2744 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.23121 \text{ kip}$$

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

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

$$M_{max} = ((-0.02828 \text{ kip/ft}) \times (60 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.2883 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2744 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2883 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2744 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2883 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2744 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.61786 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.013 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.85 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.85 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.013 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023948$</p>	<p>Status: PASS Ratio: 0.000</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>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.013 \text{ kip} \rightarrow 10013 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(10013 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.45 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.45 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.45 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.45 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.15 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.825 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.825 \text{ kip})}{(152.15 \text{ kip})}$$

$$Ratio = 0.071146$$

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

Considering z-direction:

$V_{max} = 0.23121 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.23121 \text{ kip})}{(152.15 \text{ kip})}$$

$$Ratio = 0.0015196$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 30.992 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(30.992 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.063574$$

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

Considering z-direction:

$M_{max} = 0.61786 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61786 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.0012674$$

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

REFERENCES	CALCULATIONS	RESULTS
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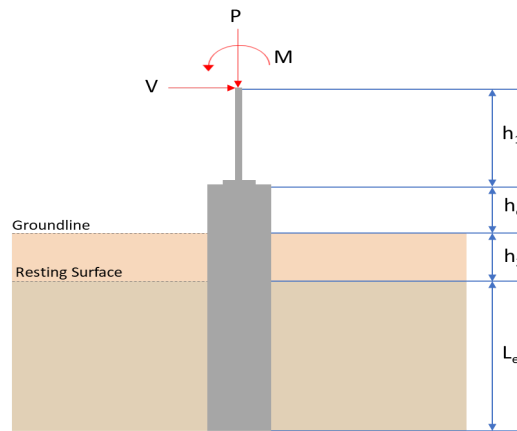
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.217	11.406
V_x (kip)	-2.922	-4.874
V_z (kip)	-0.011	-0.020
M_x (kipft)	-0.033	-0.060
M_z (kipft)	32.611	55.269

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,

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

$$H_o = \frac{(-2.922 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.37223 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.611 \text{ kipft}) + ((-2.922 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.1543 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.8619 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.011 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0014013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((-0.011 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0042038 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.65498 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8619 \text{ ft}), (0.65498 \text{ ft})]$$

$$L_{e,req} = 5.862 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.862 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.93792$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.217 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.28868 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28868 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14434$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.37223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.1543 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1543 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.1543 \text{ kipft/ft})) + (4 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3083 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1543 \text{ kipft/ft})) + (3 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.1543 \text{ kipft/ft})) + (2 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22835 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.1543 \text{ kipft/ft})) + ((-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3083 \text{ ft})}{2}$$

$$p_a = 0.32312 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22835 \text{ kip/ft}^2)}{(0.32312 \text{ kip/ft}^2)}$$

$$Ratio = 0.70672$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.91885 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.98011$$

Status: **PASS**
Ratio: **0.710**

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

Considering z-direction:

$H_o = -0.0014013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0042038 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0042038 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0042038 \text{ kipft/ft})) + (4 \times (-0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4695 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0042038 \text{ kipft/ft})) + (3 \times (-0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.0042038 \text{ kipft/ft})) + (2 \times (-0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.00035024 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.0042038 \text{ kipft/ft})) + ((-0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.000053809 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4695 \text{ ft})}{2}$$

$$p_a = 0.33521 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00035024 \text{ kip/ft}^2)}{(0.33521 \text{ kip/ft}^2)}$$

$$Ratio = -0.0010448$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

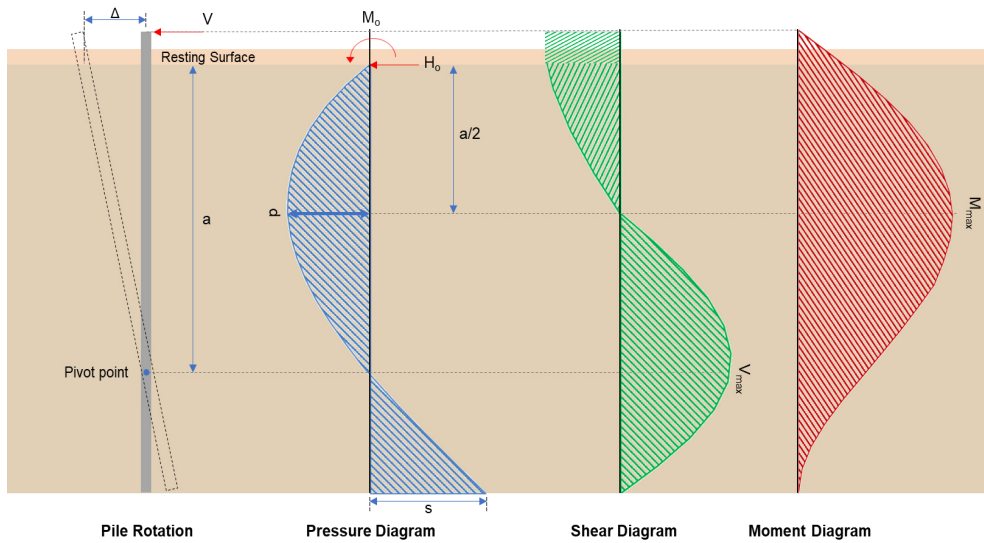
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.000053809 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = -0.000057396$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.874 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.62089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.269 \text{ kipft}) + ((-4.874 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 7.0406 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.0406 \text{ kipft/ft})}{(-0.62089 \text{ kip/ft})}$$

$$E = 11.34 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0406 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.0406 \text{ kipft/ft})) + (4 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.62089 \text{ kip/ft}) \times (60 \text{ in})}{(6 \times (7.0406 \text{ kipft/ft})) + (4 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3066 \text{ ft}$$

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

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

$$V_{max} = ((-0.62089 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3066 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3066 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.015 \text{ kip}$$

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

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

$$M_{max} = ((-0.62089 \text{ kip/ft}) \times (60 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(11.34 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 35.784 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.02 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0025478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((-0.02 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0076433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0076433 \text{ kipft/ft})}{(-0.0025478 \text{ kip/ft})}$$

$$E = 3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0076433 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0025478 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0076433 \text{ kipft/ft})) + (4 \times (-0.0025478 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4695 \text{ ft}$$

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

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

$$V_{max} = ((-0.0025478 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019313 \text{ kip}$$

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

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

$$M_{max} = ((-0.0025478 \text{ kip/ft}) \times (60 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.053255 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{11.406 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.8 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.8 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.406 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.002728$</p>	<p>Status: PASS Ratio: 0.000</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>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.406 \text{ kip} \rightarrow 11406 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(11406 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.64 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.64 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.64 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.64 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.27 \text{ kip}$$

Considering x-direction:

V_{max} = 12.015 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.015 \text{ kip})}{(152.27 \text{ kip})}$$

$$Ratio = 0.078906$$

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

Considering z-direction:

$V_{max} = 0.019313 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.019313 \text{ kip})}{(152.27 \text{ kip})}$$

$$Ratio = 0.00012683$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 35.784 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(35.784 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.073402$$

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

Considering z-direction:

$M_{max} = 0.053255 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.053255 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00010924$$

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

REFERENCES	CALCULATIONS	RESULTS
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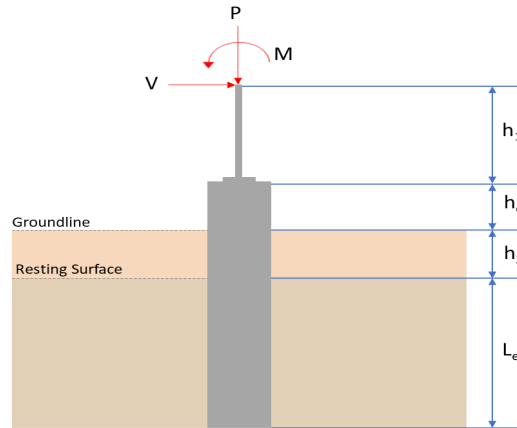
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.217	11.406
V_x (kip)	-2.922	-4.874
V_z (kip)	0.011	0.020
M_x (kipft)	0.033	0.060
M_z (kipft)	32.611	55.269

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,

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

$$H_o = \frac{(-2.922 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.37223 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.611 \text{ kipft}) + ((-2.922 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.1543 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.8619 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.011 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0014013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((0.011 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0042038 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.73544 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8619 \text{ ft}), (0.73544 \text{ ft})]$$

$$L_{e,req} = 5.862 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.862 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.93792$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.217 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.28868 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28868 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14434$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.37223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.1543 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1543 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.1543 \text{ kipft/ft})) + (4 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3083 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1543 \text{ kipft/ft})) + (3 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.1543 \text{ kipft/ft})) + (2 \times (-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22835 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.1543 \text{ kipft/ft})) + ((-0.37223 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3083 \text{ ft})}{2}$$

$$p_a = 0.32312 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22835 \text{ kip/ft}^2)}{(0.32312 \text{ kip/ft}^2)}$$

$$Ratio = 0.70672$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.91885 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.98011$$

Status: **PASS**
Ratio: **0.710**

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

Considering z-direction:

$H_o = 0.0014013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0042038 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0042038 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0042038 \text{ kipft/ft})) + (4 \times (0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4695 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0042038 \text{ kipft/ft})) + (3 \times (0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0042038 \text{ kipft/ft})) + (2 \times (0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0011832 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0042038 \text{ kipft/ft})) + ((0.0014013 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0026366 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4695 \text{ ft})}{2}$$

$$p_a = 0.33521 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0011832 \text{ kip/ft}^2)}{(0.33521 \text{ kip/ft}^2)}$$

$$Ratio = 0.0035299$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

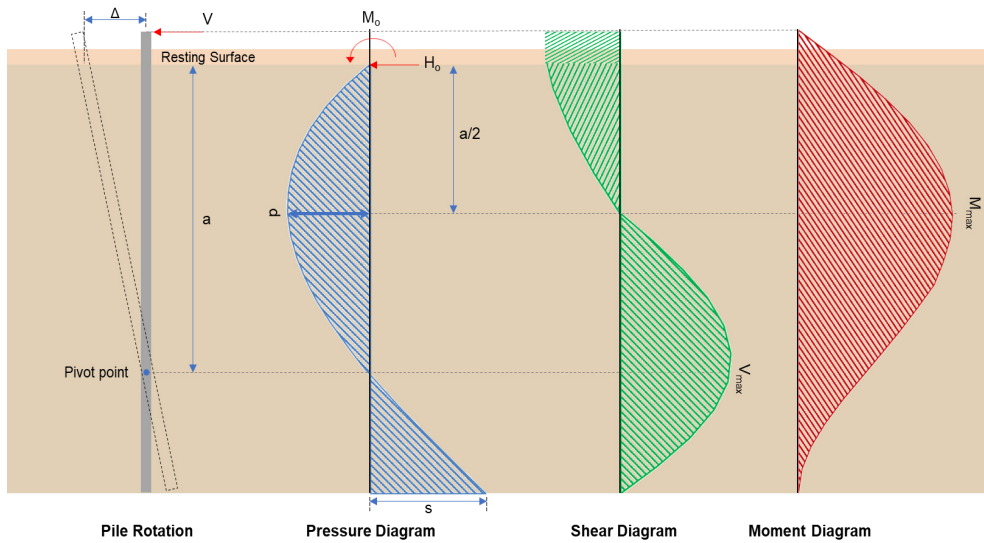
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0026366 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0028124$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.874 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.62089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.269 \text{ kipft}) + ((-4.874 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 7.0406 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.0406 \text{ kipft/ft})}{(-0.62089 \text{ kip/ft})}$$

$$E = 11.34 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0406 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.0406 \text{ kipft/ft})) + (4 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.62089 \text{ kip/ft}) \times (60 \text{ in})}{(6 \times (7.0406 \text{ kipft/ft})) + (4 \times (-0.62089 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3066 \text{ ft}$$

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

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

$$V_{max} = ((-0.62089 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3066 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3066 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.015 \text{ kip}$$

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

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

$$M_{max} = ((-0.62089 \text{ kip/ft}) \times (60 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(11.34 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.34 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3066 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 35.784 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.02 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0025478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((0.02 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0076433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0076433 \text{ kipft/ft})}{(0.0025478 \text{ kip/ft})}$$

$$E = 3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0076433 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0025478 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0076433 \text{ kipft/ft})) + (4 \times (0.0025478 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4695 \text{ ft}$$

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

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

$$V_{max} = ((0.0025478 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019313 \text{ kip}$$

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

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

$$M_{max} = ((0.0025478 \text{ kip/ft}) \times (60 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.053255 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{11.406 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.8 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.8 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

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;">$Ratio = 0.96007$</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>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.406 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.002728$</p>	<p>Status: PASS Ratio: 0.000</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>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.406 \text{ kip} \rightarrow 11406 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(11406 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.64 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.64 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.64 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.64 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.27 \text{ kip}$$

Considering x-direction:

V_{max} = 12.015 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.015 \text{ kip})}{(152.27 \text{ kip})}$$

$$Ratio = 0.078906$$

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

Considering z-direction:

$V_{max} = 0.019313 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.019313 \text{ kip})}{(152.27 \text{ kip})}$$

$$Ratio = 0.00012683$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 35.784 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(35.784 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.073402$$

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

Considering z-direction:

$M_{max} = 0.053255 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.053255 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00010924$$

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