

Site Details:



Site Address: Glendive, MT 59330, USA

Array Specification

Duty Classification:	HD
Module Width:	41.50 in
Module Length:	68.80in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	18.41 ft
Total Frame Length:	68.75 ft
Module Info/Notes:	
Array Dimensions N/S:	17.50 ft
Array Dimensions E/W:	69.80 ft
Rail Length:	210.00 in
Rail Spacing:	2.91 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.70 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 7.25 ft Pile 3: 7.25 ft Pile 4: 6.75 ft
Foundation Volume:	16.593 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Glendive, MT 59330, USA
Wind Speed:	105 mph

Snow Load:

16 psf

Design Disclaimer

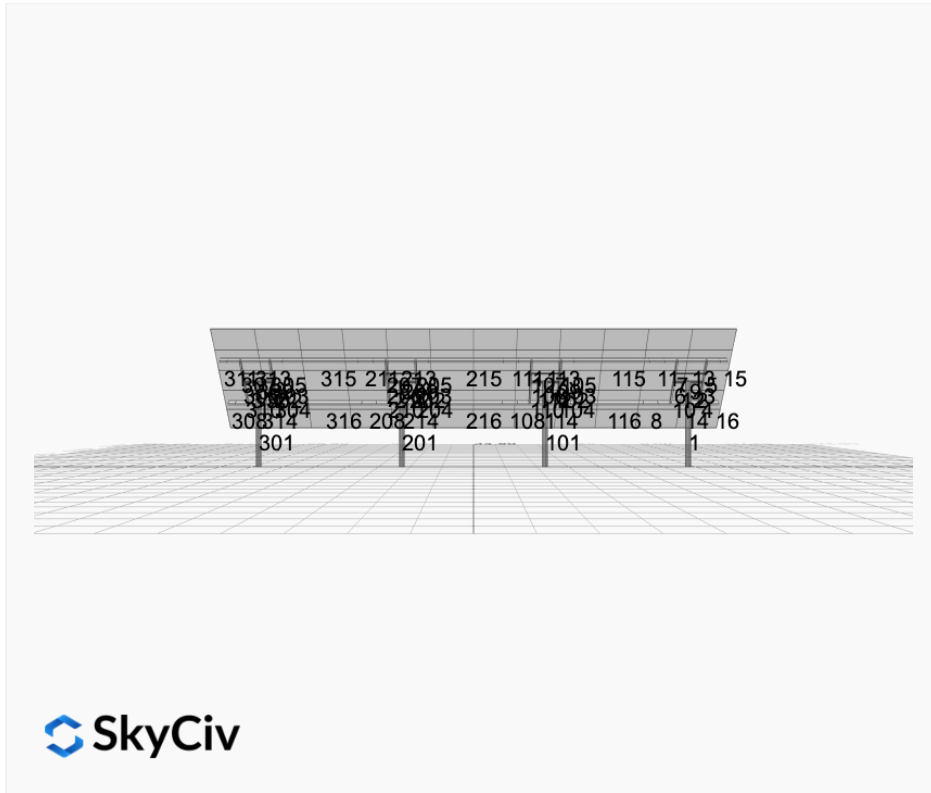
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

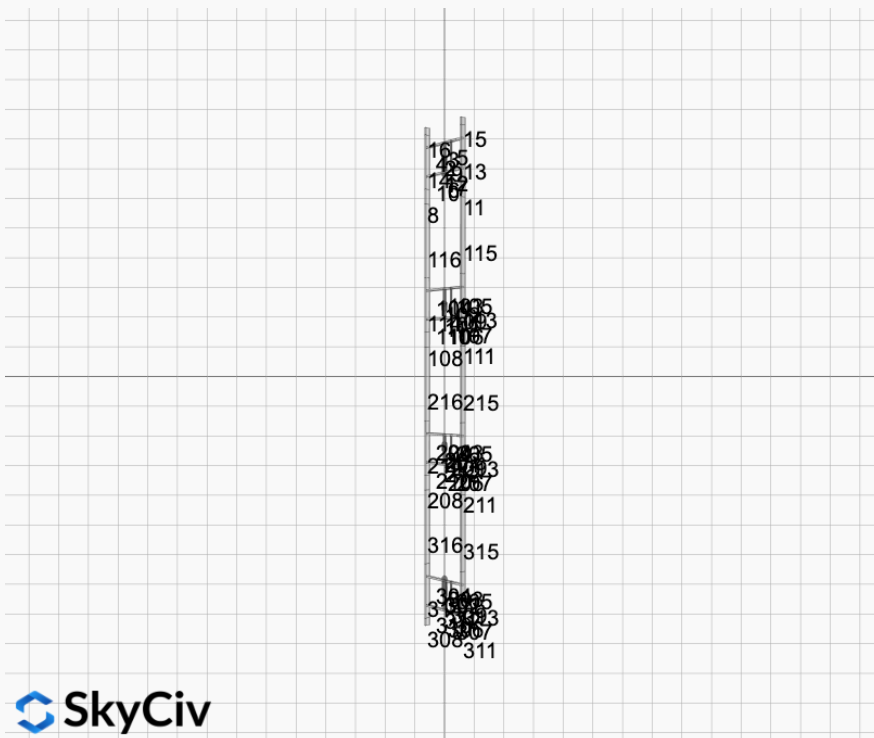
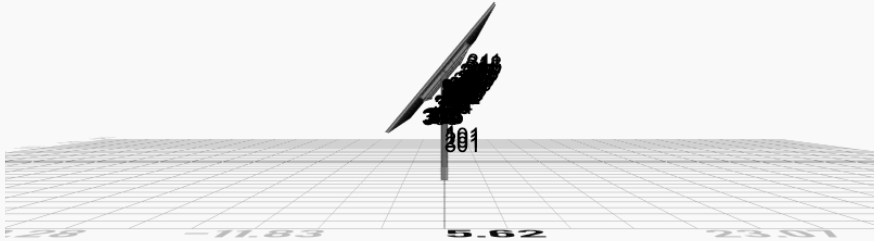
AutoDesigner Input

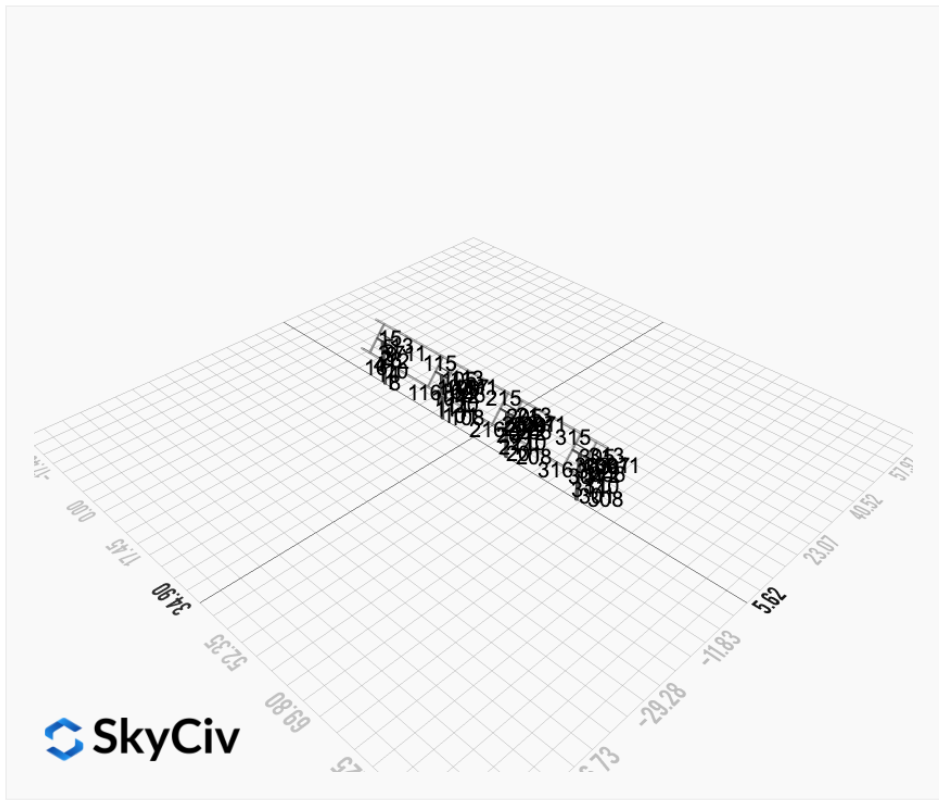
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Design Notes:

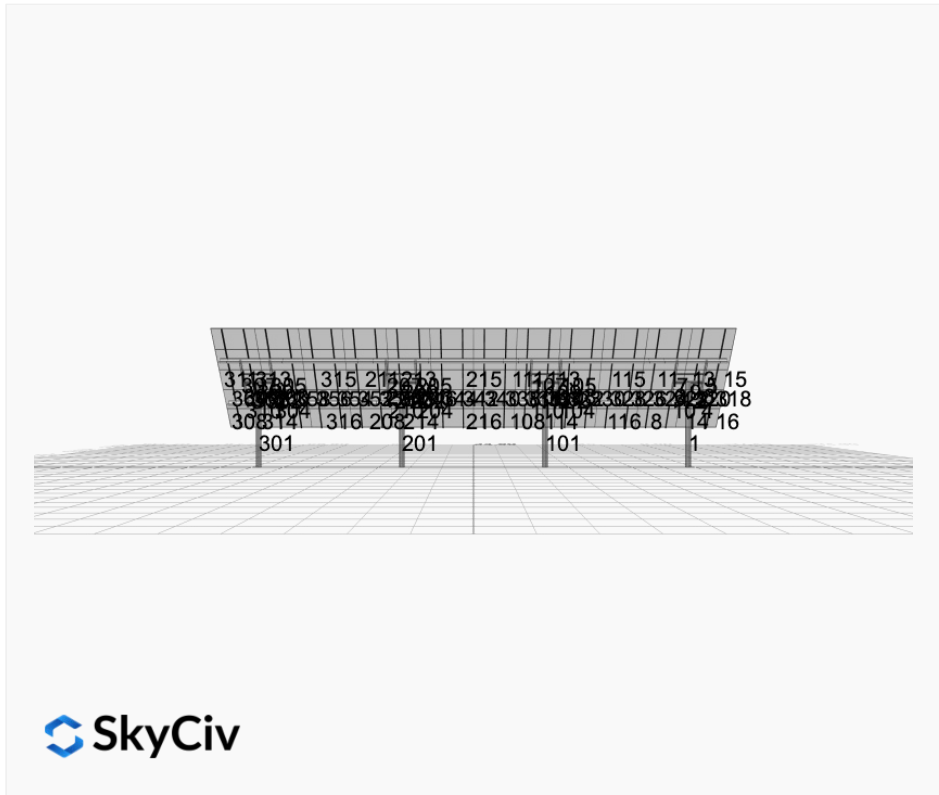
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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







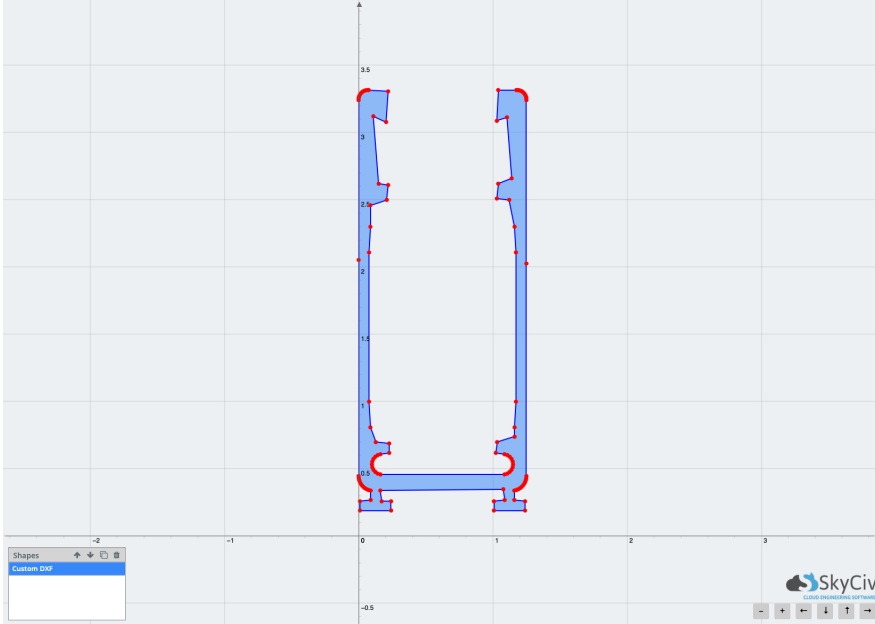
 SkyCiv



 SkyCiv

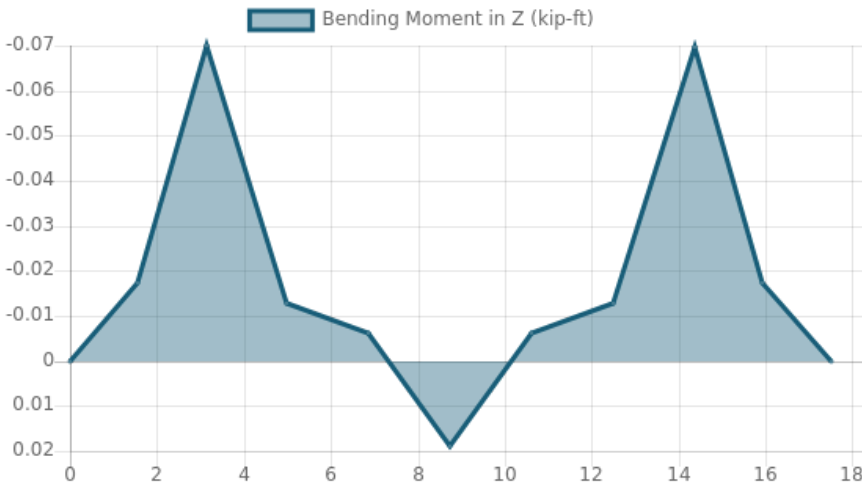
Rail Design Check

Rail Length: 17.5 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 2.90833333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0066 kip/ft
Snow (Y): -0.0078 kip/ft
Wind uplift Case A: 0.0645 kip/ft
Wind downforce Case A: 0.0645 kip/ft
Dead (Panel load) (X): 0.0086 kip/ft
Dead (Panel load) (Y): -0.0102 kip/ft

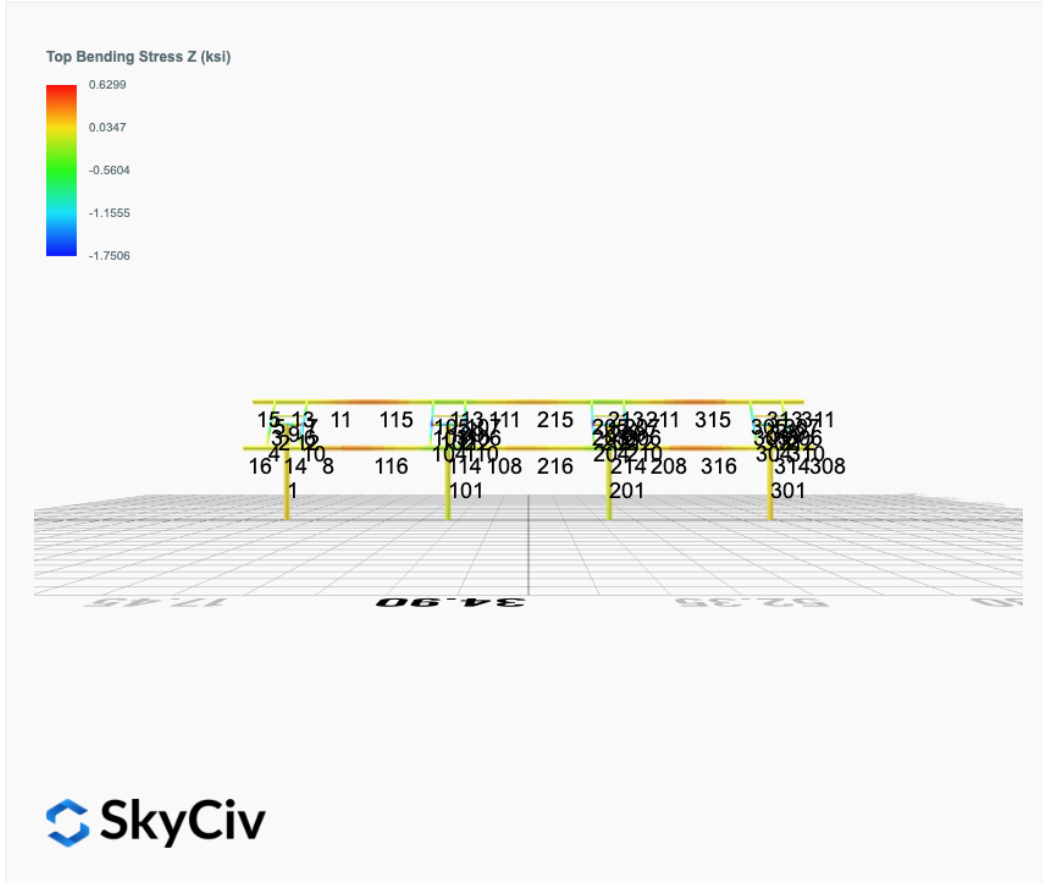
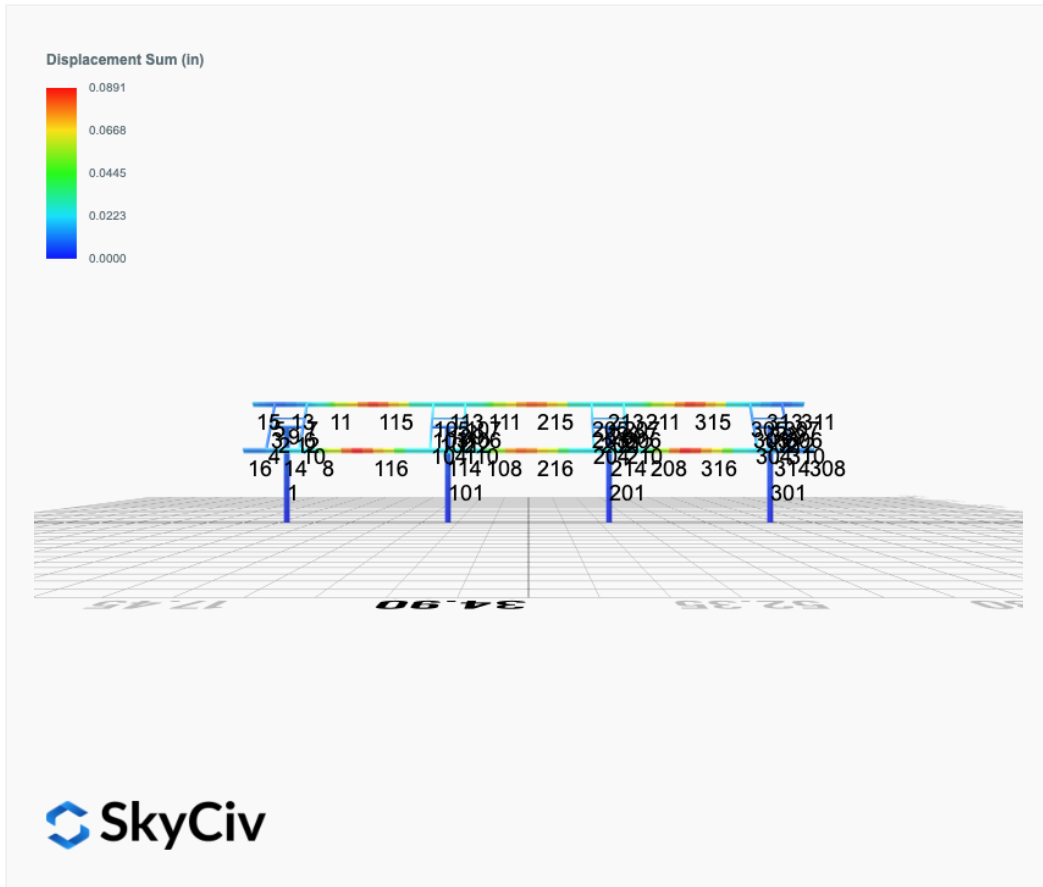


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	8.52936292	0.247	PASS
Material Yield	34.5	8.52936292	0.247	PASS
Material Strength	37	8.52936292	0.231	PASS

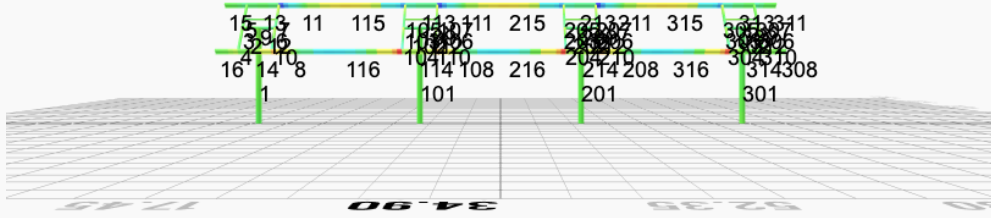
Member 1, ULS: 1. 1.4D



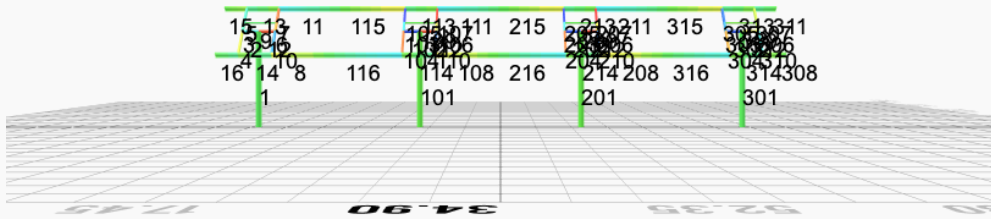
FEM Results (Envelope Worst Case for each member)



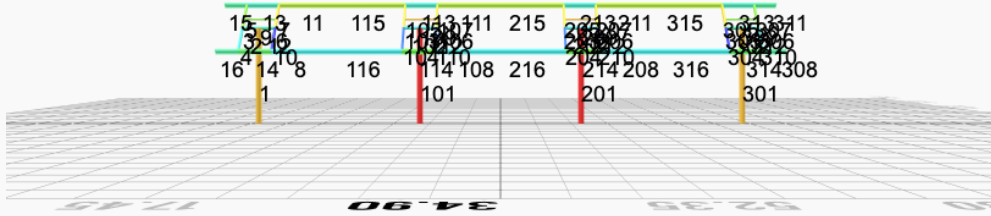
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 2. D + L	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 3. D + (S or Lr or R)	0.0494	2.5644	0.1335	0.4701	-0.1757	-0.5254
ULS: 3. D + (S or Lr or R)	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0459	2.4230	0.1242	0.4372	-0.1633	-0.4872
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 5b. D + 0.7E	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0459	2.4230	0.1242	0.4372	-0.1633	-0.4872
ULS: 8. 0.6D + 0.7E	0.0213	1.1994	0.0577	0.2030	-0.0758	-0.2236
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5719	4.1255	0.3857	1.2995	-1.6038	30.8324
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6373	-0.1244	-0.1868	-0.5992	1.3181	-30.8012
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9096	4.0179	0.3414	1.1580	-1.2714	22.9166
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0459	2.4230	0.1242	0.4372	-0.1633	-0.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9972	0.8305	-0.0880	-0.2660	0.9200	-23.3086
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0459	2.4230	0.1242	0.4372	-0.1633	-0.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9200	3.5939	0.3133	1.0592	-1.2344	23.0311
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9869	0.4064	-0.1161	-0.3648	0.9570	-23.1941
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0355	1.9989	0.0961	0.3384	-0.1264	-0.3727
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5861	3.3259	0.3473	1.1641	-1.5532	30.9814
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0213	1.1994	0.0577	0.2030	-0.0758	-0.2236
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6231	-0.9240	-0.2252	-0.7346	1.3687	-30.6521
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0213	1.1994	0.0577	0.2030	-0.0758	-0.2236

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	6.2268
Shear X	-4.3886
Shear Z	0.6204
Moment X	2.0868
Moment Y (Twist)	2.6557
Moment Z	52.0239

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	4.1255
Shear X	-2.6373
Shear Z	0.3857
Moment X	1.2995
Moment Y (Twist)	1.6038
Moment Z	30.9814

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	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 2. D + L	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 3. D + (S or Lr or R)	-0.0494	3.3843	-0.0052	-0.0191	0.0232	0.5698
ULS: 3. D + (S or Lr or R)	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0459	3.1855	-0.0048	-0.0178	0.0216	0.5313

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 5b. D + 0.7E	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0459	3.1855	-0.0048	-0.0178	0.0216	0.5313
ULS: 8. 0.6D + 0.7E	-0.0213	1.5534	-0.0022	-0.0082	0.0100	0.2494
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6576	5.6896	0.0278	0.0864	-0.2032	43.1519
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5921	-0.5147	-0.0314	-0.1013	0.2158	-41.0539
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7624	5.5109	0.0188	0.0574	-0.1433	32.5834
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0459	3.1855	-0.0048	-0.0178	0.0216	0.5313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6748	0.8576	-0.0256	-0.0835	0.1709	-30.5710
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0459	3.1855	-0.0048	-0.0178	0.0216	0.5313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7520	4.9145	0.0199	0.0614	-0.1482	32.4679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6852	0.2612	-0.0245	-0.0794	0.1660	-30.6865
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0355	2.5891	-0.0037	-0.0137	0.0167	0.4157
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6434	4.6540	0.0293	0.0919	-0.2098	42.9856
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0213	1.5534	-0.0022	-0.0082	0.0100	0.2494
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6063	-1.5503	-0.0299	-0.0958	0.2091	-41.2202
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0213	1.5534	-0.0022	-0.0082	0.0100	0.2494

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.6710
Shear X	-6.0831
Shear Z	-0.0533
Moment X	-0.1714
Moment Y (Twist)	0.3664
Moment Z	72.6294

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.6896
Shear X	-3.6576
Shear Z	-0.0314
Moment X	-0.1013
Moment Y (Twist)	0.2158
Moment Z	43.1519

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	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 2. D + L	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 3. D + (S or Lr or R)	-0.0494	3.3843	0.0052	0.0191	-0.0232	0.5698
ULS: 3. D + (S or Lr or R)	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0459	3.1855	0.0048	0.0177	-0.0216	0.5313
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 5b. D + 0.7E	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0459	3.1855	0.0048	0.0177	-0.0216	0.5313
ULS: 8. 0.6D + 0.7E	-0.0213	1.5534	0.0022	0.0082	-0.0100	0.2494
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6576	5.6896	-0.0278	-0.0865	0.2032	43.1519
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5921	-0.5147	0.0314	0.1013	-0.2158	-41.0539
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157

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	-2.7624	5.5109	-0.0188	-0.0574	0.1433	32.5834
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0459	3.1855	0.0048	0.0177	-0.0216	0.5313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6748	0.8576	0.0256	0.0834	-0.1709	-30.5710
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0459	3.1855	0.0048	0.0177	-0.0216	0.5313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7520	4.9145	-0.0199	-0.0614	0.1482	32.4679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6852	0.2612	0.0245	0.0794	-0.1660	-30.6865
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0355	2.5891	0.0037	0.0137	-0.0167	0.4157
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6434	4.6540	-0.0293	-0.0920	0.2098	42.9856
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0213	1.5534	0.0022	0.0082	-0.0100	0.2494
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6063	-1.5503	0.0299	0.0958	-0.2091	-41.2202
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0213	1.5534	0.0022	0.0082	-0.0100	0.2494

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.6710
Shear X	-6.0831
Shear Z	0.0533
Moment X	0.1719
Moment Y (Twist)	0.3664
Moment Z	72.6296

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.6896
Shear X	-3.6576
Shear Z	0.0314
Moment X	0.1013
Moment Y (Twist)	0.2158
Moment Z	43.1519

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	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 2. D + L	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 3. D + (S or Lr or R)	0.0494	2.5644	-0.1335	-0.4702	0.1757	-0.5254
ULS: 3. D + (S or Lr or R)	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0459	2.4230	-0.1242	-0.4372	0.1634	-0.4872
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 5b. D + 0.7E	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0459	2.4230	-0.1242	-0.4372	0.1634	-0.4872
ULS: 8. 0.6D + 0.7E	0.0213	1.1994	-0.0577	-0.2030	0.0758	-0.2236
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5719	4.1255	-0.3857	-1.2995	1.6038	30.8324
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6373	-0.1244	0.1868	0.5992	-1.3181	-30.8012
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9096	4.0179	-0.3414	-1.1581	1.2714	22.9166
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0459	2.4230	-0.1242	-0.4372	0.1634	-0.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9972	0.8305	0.0880	0.2659	-0.9200	-23.3086
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0459	2.4230	-0.1242	-0.4372	0.1634	-0.4872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9200	3.5939	-0.3133	-1.0592	1.2344	23.0311
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9869	0.4064	0.1161	0.3648	-0.9570	-23.1941
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0355	1.9989	-0.0961	-0.3384	0.1264	-0.3727

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5861	3.3259	-0.3473	-1.1642	1.5532	30.9815
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0213	1.1994	-0.0577	-0.2030	0.0758	-0.2236
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6231	-0.9240	0.2252	0.7345	-1.3686	-30.6521
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0213	1.1994	-0.0577	-0.2030	0.0758	-0.2236

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.2268
Shear X	-4.3886
Shear Z	-0.6204
Moment X	-2.0873
Moment Y (Twist)	2.6560
Moment Z	52.0248

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	4.1255
Shear X	-2.6373
Shear Z	-0.3857
Moment X	-1.2995
Moment Y (Twist)	1.6038
Moment Z	30.9815

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: Ray Pasta 5x12 - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions							

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

Section Dimensions							

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	

Section Dimensions							

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

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

314	19	4.88	4.00	0	2,1.37,1.50,1.37,1.54,1.37	0	0	1
315	19	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.10,1.08,1.09,1.08,1.10,1.08,1.08,1.08,1.13,1.08,1.09,1.08,1.11,1.08,1.09,1.08,1.10,1.08	30.0	20.0	1
316	19	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.07,1.08,1.08,1.08,1.08	30.0	20.0	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	180.90	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	123.95	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	26.69	6.12	40.24	43.62
14	133.20	85.85	31.49	6.12	40.24	43.62
15	133.20	121.82	32.87	6.12	40.24	43.62
16	133.20	121.82	32.87	6.12	40.24	43.62
101	377.97	180.90	83.29	83.29	113.39	113.39
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	24.37	6.12	40.24	43.62
114	133.20	85.85	24.26	6.12	40.24	43.62
115	133.20	69.16	16.48	6.12	40.24	43.62
116	133.20	69.16	17.48	6.12	40.24	43.62
201	377.97	180.90	83.29	83.29	113.39	113.39
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62

212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	24.37	6.12	40.24	43.62
214	133.20	85.85	24.26	6.12	40.24	43.62
215	133.20	69.16	16.76	6.12	40.24	43.62
216	133.20	69.16	17.58	6.12	40.24	43.62
301	377.97	180.90	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	121.82	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	121.82	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	26.69	6.12	40.24	43.62
314	133.20	85.85	31.48	6.12	40.24	43.62
315	133.20	69.16	16.72	6.12	40.24	43.62
316	133.20	69.16	16.59	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	0.034	0.625	0.062	0.039	0.005	0.667	#13	0.502	Not Required	Pass
2	0.001	0.097	0.121	0.031	0.028	0.219	#13	0.053	Not Required	Pass
3	0.003	0.295	0.019	0.028	0.003	0.301	#13	0.045	Not Required	Pass
4	0.003	0.298	0.065	0.030	0.014	0.364	#13	0.120	Not Required	Pass
5	0.003	0.183	0.016	0.029	0.005	0.189	#13	0.074	Not Required	Pass
6	0.007	0.553	0.066	0.058	0.018	0.610	#13	0.045	Not Required	Pass
7	0.007	0.343	0.106	0.055	0.025	0.359	#13	0.074	Not Required	Pass
8	0.006	0.122	0.105	0.032	0.010	0.180	#13	0.095	Not Required	Pass
9	0.003	0.060	0.070	0.005	0.004	0.120	#13	0.204	Not Required	Pass
10	0.008	0.508	0.119	0.051	0.028	0.522	#13	0.080	Not Required	Pass
11	0.004	0.107	0.104	0.036	0.010	0.159	#13	0.095	Not Required	Pass
12	0.002	0.333	0.242	0.067	0.049	0.576	#13	0.053	Not Required	Pass
13	0.006	0.070	0.251	0.047	0.013	0.270	#23	0.286	Not Required	Pass
14	0.006	0.056	0.251	0.043	0.013	0.256	#23	0.190	Not Required	Pass
15	0.000	0.004	0.006	0.006	0.002	0.008	#21	Not Required	Not Required	Pass
16	0.000	0.004	0.006	0.006	0.002	0.008	#21	Not Required	Not Required	Pass
101	0.048	0.872	0.005	0.054	0.000	0.898	#13	0.502	Not Required	Pass
102	0.002	0.298	0.256	0.069	0.049	0.555	#13	0.035	Not Required	Pass
103	0.007	0.563	0.041	0.056	0.006	0.599	#13	0.045	Not Required	Pass
104	0.007	0.596	0.111	0.060	0.024	0.666	#13	0.080	Not Required	Pass
105	0.007	0.348	0.113	0.056	0.028	0.372	#13	0.074	Not Required	Pass
106	0.006	0.600	0.042	0.060	0.008	0.628	#13	0.045	Not Required	Pass
107	0.006	0.373	0.102	0.060	0.026	0.396	#13	0.074	Not Required	Pass
108	0.007	0.045	0.088	0.037	0.010	0.110	#21	0.095	Not Required	Pass
109	0.009	0.044	0.048	0.001	0.000	0.096	#13	0.204	Not Required	Pass
110	0.006	0.601	0.007	0.060	0.021	0.640	#13	0.080	Not Required	Pass

110	0.000	0.001	0.097	0.000	0.021	0.049	#13	0.000	Not Required	Pass
111	0.004	0.073	0.090	0.037	0.010	0.100	#23	0.095	Not Required	Pass
112	0.002	0.317	0.272	0.070	0.053	0.591	#13	0.035	Not Required	Pass
113	0.006	0.134	0.261	0.048	0.013	0.333	#21	0.286	Not Required	Pass
114	0.010	0.188	0.262	0.052	0.013	0.392	#13	0.286	Not Required	Pass
115	0.007	0.318	0.129	0.038	0.010	0.419	#13	0.473	Not Required	Pass
116	0.006	0.297	0.131	0.042	0.011	0.401	#13	0.473	Not Required	Pass
201	0.048	0.872	0.005	0.054	0.000	0.898	#13	0.502	Not Required	Pass
202	0.002	0.317	0.273	0.070	0.053	0.591	#13	0.035	Not Required	Pass
203	0.006	0.600	0.042	0.060	0.008	0.628	#13	0.045	Not Required	Pass
204	0.006	0.601	0.097	0.060	0.021	0.649	#13	0.080	Not Required	Pass
205	0.006	0.373	0.102	0.060	0.026	0.396	#13	0.074	Not Required	Pass
206	0.007	0.563	0.041	0.056	0.006	0.599	#13	0.045	Not Required	Pass
207	0.007	0.348	0.113	0.056	0.028	0.372	#13	0.074	Not Required	Pass
208	0.006	0.074	0.113	0.042	0.011	0.130	#13	0.095	Not Required	Pass
209	0.009	0.044	0.048	0.001	0.000	0.096	#13	0.204	Not Required	Pass
210	0.007	0.596	0.111	0.060	0.024	0.666	#13	0.080	Not Required	Pass
211	0.004	0.100	0.113	0.038	0.010	0.133	#13	0.095	Not Required	Pass
212	0.002	0.298	0.256	0.069	0.049	0.555	#13	0.035	Not Required	Pass
213	0.006	0.134	0.261	0.048	0.013	0.333	#21	0.286	Not Required	Pass
214	0.010	0.188	0.262	0.052	0.013	0.392	#13	0.286	Not Required	Pass
215	0.008	0.254	0.130	0.037	0.010	0.353	#13	0.473	Not Required	Pass
216	0.008	0.194	0.130	0.037	0.010	0.298	#13	0.473	Not Required	Pass
301	0.034	0.625	0.062	0.039	0.005	0.667	#13	0.502	Not Required	Pass
302	0.002	0.333	0.242	0.067	0.049	0.576	#13	0.053	Not Required	Pass
303	0.007	0.553	0.066	0.058	0.018	0.610	#13	0.045	Not Required	Pass
304	0.008	0.508	0.119	0.051	0.028	0.522	#13	0.080	Not Required	Pass
305	0.007	0.343	0.106	0.055	0.025	0.359	#13	0.074	Not Required	Pass
306	0.003	0.295	0.019	0.028	0.003	0.301	#13	0.045	Not Required	Pass
307	0.003	0.183	0.016	0.029	0.005	0.189	#13	0.074	Not Required	Pass
308	0.000	0.004	0.006	0.006	0.002	0.008	#21	Not Required	Not Required	Pass
309	0.003	0.060	0.070	0.005	0.004	0.120	#13	0.204	Not Required	Pass
310	0.003	0.298	0.065	0.030	0.014	0.363	#13	0.120	Not Required	Pass
311	0.000	0.004	0.006	0.006	0.002	0.008	#21	Not Required	Not Required	Pass
312	0.001	0.097	0.121	0.031	0.028	0.219	#13	0.053	Not Required	Pass
313	0.006	0.070	0.251	0.047	0.013	0.270	#23	0.190	Not Required	Pass
314	0.006	0.056	0.251	0.043	0.013	0.256	#23	0.286	Not Required	Pass
315	0.007	0.321	0.131	0.036	0.010	0.418	#13	0.473	Not Required	Pass
316	0.006	0.318	0.129	0.032	0.010	0.424	#13	0.473	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS
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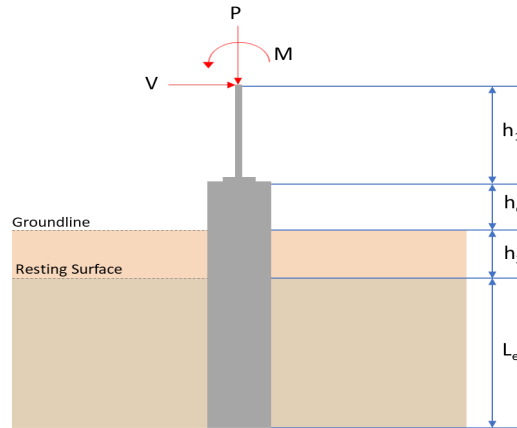
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

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)	4.125	6.227
V_x (kip)	-2.637	-4.389
V_z (kip)	0.386	0.620
M_x (kipft)	1.299	2.087
M_z (kipft)	30.981	52.024

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,

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

$$H_o = \frac{(-2.637 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4199 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.981 \text{ kipft}) + ((-2.637 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.2009 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.386 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.061465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.299 \text{ kipft}) + ((0.386 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20685 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.0259 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.2009 \text{ ft}), (3.0259 \text{ ft})]$$

$$L_{e,req} = 6.201 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.201 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.91867$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.125 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.25781 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.25781 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12891$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.4199 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.9333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9333 \text{ kipft/ft})) + (4 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6558 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9333 \text{ kipft/ft})) + (3 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9333 \text{ kipft/ft})) + (2 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.22735 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.9333 \text{ kipft/ft})) + ((-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.92605 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6558 \text{ ft})}{2}$$

$$p_a = 0.34918 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22735 \text{ kip/ft}^2)}{(0.34918 \text{ kip/ft}^2)}$$

$$Ratio = 0.65109$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.92605 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.91462$$

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

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

Considering z-direction:

$H_o = 0.061465 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.20685 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20685 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.061465 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.20685 \text{ kipft/ft})) + (4 \times (0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8218 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.20685 \text{ kipft/ft})) + (3 \times (0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.20685 \text{ kipft/ft})) + (2 \times (0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.04873 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.20685 \text{ kipft/ft})) + ((0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.10911 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8218 \text{ ft})}{2}$$

$$p_a = 0.36164 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.04873 \text{ kip/ft}^2)}{(0.36164 \text{ kip/ft}^2)}$$

$$Ratio = 0.13475$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

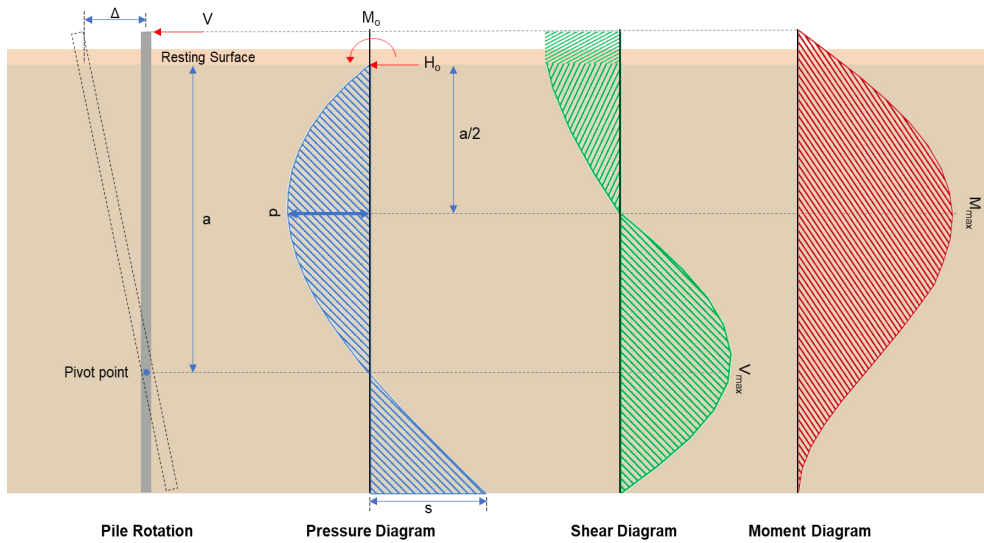
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.10911 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.10777$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.389 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.69889 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.024 \text{ kipft}) + ((-4.389 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.2841 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.2841 \text{ kipft/ft})}{(-0.69889 \text{ kip/ft})}$$

$$E = 11.853 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.2841 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.2841 \text{ kipft/ft})) + (4 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(-0.69889 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (8.2841 \text{ kip/ft})) + (4 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6548 \text{ ft}$$

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

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

$$V_{max} = ((-0.69889 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6548 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6548 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.531 \text{ kip}$$

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

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

$$M_{max} = ((-0.69889 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.853 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.827 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.62 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.098726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.087 \text{ kipft}) + ((0.62 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.33232 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.33232 \text{ kipft/ft})}{(0.098726 \text{ kip/ft})}$$

$$E = 3.3661 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.33232 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.098726 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.33232 \text{ kipft/ft})) + (4 \times (0.098726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8218 \text{ ft}$$

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

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

$$V_{max} = ((0.098726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8218 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8218 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.6116 \text{ kip}$$

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

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

$$M_{max} = ((0.098726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.3661 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.8264 \text{ kipft}$$

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,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.227 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.389 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.389 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

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;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</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>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</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;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.227 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023277$</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>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.227 \text{ kip} \rightarrow 6227 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6227 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.32 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.32 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.32 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.32 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.64 \text{ kip}$$

Considering x-direction:

V_{max} = 10.531 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.531 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.095183$$

Considering z-direction:

$V_{max} = 0.6116 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.6116 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.005528$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 33.827 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(33.827 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13553$$

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

Considering z-direction:

$M_{max} = 1.8264 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.8264 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0073173$$

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

REFERENCES	CALCULATIONS	RESULTS
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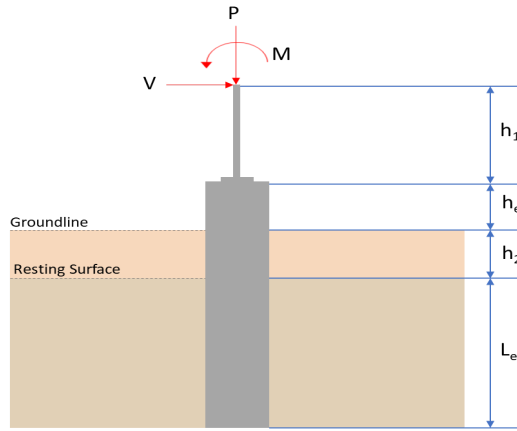
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
 $b = 48$ in - Pile width
 $D = 48$ in - Pile depth
 $L = 6.75$ ft - Total pile length
 $h_1 = 0$ ft - Lateral load height from the top of the pile,
 $h_2 = 0$ ft - Depth to resisting surface
 $h_e = 0$ ft - Length of pile above the ground

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)	4.125	6.227
V_x (kip)	-2.637	-4.389
V_z (kip)	-0.386	-0.620
M_x (kipft)	-1.300	-2.087
M_z (kipft)	30.981	52.025

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,

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

$$H_o = \frac{(-2.637 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4199 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.981 \text{ kipft}) + ((-2.637 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.2009 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.386 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.061465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.3 \text{ kipft}) + ((-0.386 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20701 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.0735 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2009 \text{ ft}), (2.0735 \text{ ft})]$$

$$L_{e,req} = 6.201 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(6.201 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.91867$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.125 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.25781 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.25781 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.12891$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.4199 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.9333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9333 \text{ kipft/ft})) + (4 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6558 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9333 \text{ kipft/ft})) + (3 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9333 \text{ kipft/ft})) + (2 \times (-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.22735 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.9333 \text{ kipft/ft})) + ((-0.4199 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.92605 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6558 \text{ ft})}{2}$$

$$p_a = 0.34918 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22735 \text{ kip/ft}^2)}{(0.34918 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65109$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.92605 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91462$$

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

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

Considering z-direction:

$H_o = -0.061465 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.20701 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20701 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.061465 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.20701 \text{ kipft/ft})) + (4 \times (-0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8217 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.20701 \text{ kipft/ft})) + (3 \times (-0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.20701 \text{ kipft/ft})) + (2 \times (-0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = -0.013688 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.20701 \text{ kipft/ft})) + ((-0.061465 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = -0.00011533 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8217 \text{ ft})}{2}$$

$$p_a = 0.36163 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.013688 \text{ kip/ft}^2)}{(0.36163 \text{ kip/ft}^2)}$$

$$Ratio = -0.03785$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

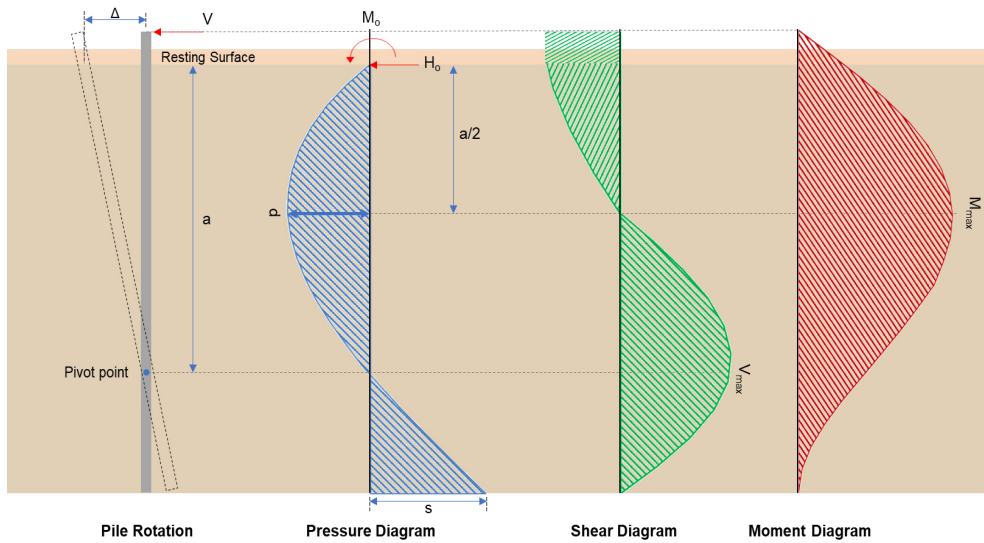
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00011533 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = -0.00011391$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.389 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.69889 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.025 \text{ kipft}) + ((-4.389 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.2842 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.2842 \text{ kipft/ft})}{(-0.69889 \text{ kip/ft})}$$

$$E = 11.853 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.2842 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.2842 \text{ kipft/ft})) + (4 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(-0.69889 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (8.2842 \text{ kipft/ft})) + (4 \times (-0.69889 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6548 \text{ ft}$$

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

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

$$V_{max} = ((-0.69889 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6548 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6548 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.531 \text{ kip}$$

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

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

$$M_{max} = ((-0.69889 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.853 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.853 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6548 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.828 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.62 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.098726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.087 \text{ kipft}) + ((-0.62 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.33232 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.33232 \text{ kipft/ft})}{(-0.098726 \text{ kip/ft})}$$

$$E = 3.3661 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.33232 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.098726 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.33232 \text{ kipft/ft})) + (4 \times (-0.098726 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8218 \text{ ft}$$

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

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

$$V_{max} = ((-0.098726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8218 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8218 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.6116 \text{ kip}$$

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

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

$$M_{max} = ((-0.098726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.3661 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3661 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8218 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.8264 \text{ kipft}$$

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,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.227 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.389 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.389 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

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;">$Ratio = 0.96556$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</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;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.227 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023277$</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>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.227 \text{ kip} \rightarrow 6227 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6227 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.32 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.32 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.32 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.32 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.64 \text{ kip}$$

Considering x-direction:

V_{max} = 10.531 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.531 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.095184$$

Considering z-direction:

$V_{max} = 0.6116 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.6116 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.005528$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 33.828 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(33.828 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13553$$

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

Considering z-direction:

$M_{max} = 1.8264 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.8264 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0073173$$

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

REFERENCES	CALCULATIONS	RESULTS
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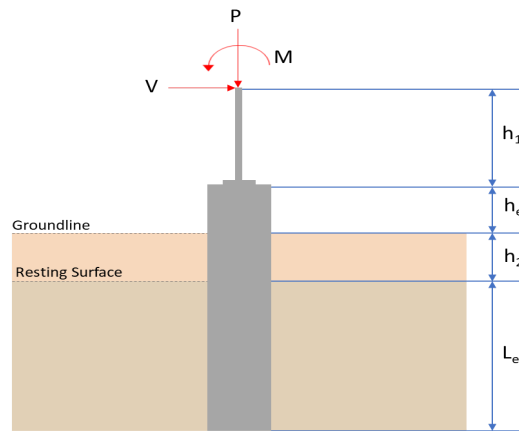
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ 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)	5.690	8.671
V_x (kip)	-3.658	-6.083
V_z (kip)	-0.031	-0.053
M_x (kipft)	-0.101	-0.171
M_z (kipft)	43.152	72.629

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,

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

$$H_o = \frac{(-3.658 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.152 \text{ kipft}) + ((-3.658 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8713 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.7863 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.031 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0049363 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.101 \text{ kipft}) + ((-0.031 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016083 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.99696 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.7863 \text{ ft}), (0.99696 \text{ ft})]$$

$$L_{e,req} = 6.786 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(6.786 \text{ ft})}{(7.25 \text{ ft})}$$

$$Ratio = 0.936$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.69 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.35563 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35563 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17781$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58248 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8713 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8713 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.8713 \text{ kipft/ft})) + (4 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0089 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8713 \text{ kipft/ft})) + (3 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.8713 \text{ kipft/ft})) + (2 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.25742 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8713 \text{ kipft/ft})) + ((-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0867 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0089 \text{ ft})}{2}$$

$$p_a = 0.37567 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.25742 \text{ kip/ft}^2)}{(0.37567 \text{ kip/ft}^2)}$$

$$Ratio = 0.68524$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0867 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.99924$$

Status: **PASS**
Ratio: **0.690**

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

Considering z-direction:

$H_o = -0.0049363 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.016083 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.016083 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.016083 \text{ kipft/ft})) + (4 \times (-0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1942 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.016083 \text{ kipft/ft})) + (3 \times (-0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.016083 \text{ kipft/ft})) + (2 \times (-0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.0011327 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.016083 \text{ kipft/ft})) + ((-0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.00041352 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1942 \text{ ft})}{2}$$

$$p_a = 0.38957 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0011327 \text{ kip/ft}^2)}{(0.38957 \text{ kip/ft}^2)}$$

$$Ratio = -0.0029076$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

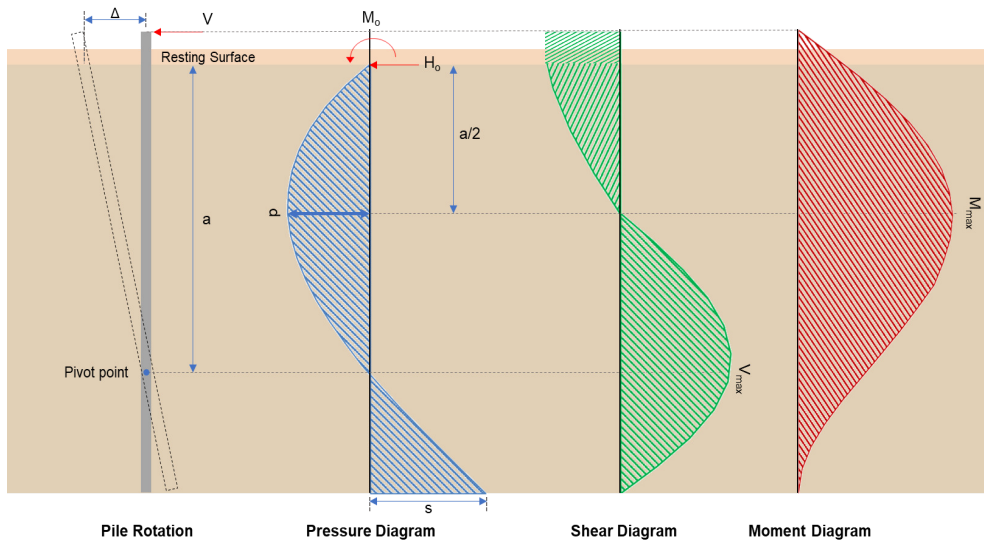
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00041352 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00038025$$

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,

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

$$H_o = \frac{(-6.083 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96863 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(72.629 \text{ kipft}) + ((-6.083 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.565 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.565 \text{ kipft/ft})}{(-0.96863 \text{ kip/ft})}$$

$$E = 11.94 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.565 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{6 \times (11.565 \text{ kipft/ft}) + 4 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft})}$$

$$a = \frac{(-0.96863 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})}{(6 \times (11.565 \text{ kipft/ft})) + (4 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0074 \text{ ft}$$

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

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

$$V_{max} = ((-0.96863 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0074 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0074 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.846 \text{ kip}$$

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

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

$$M_{max} = ((-0.96863 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(11.94 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 47.642 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.053 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0084395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.171 \text{ kipft}) + ((-0.053 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.027229 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.027229 \text{ kipft/ft})}{(-0.0084395 \text{ kip/ft})}$$

$$E = 3.2264 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.027229 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0084395 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.027229 \text{ kipft/ft})) + (4 \times (-0.0084395 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1956 \text{ ft}$$

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

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

$$V_{max} = ((-0.0084395 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2264 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1956 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2264 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1956 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.049115 \text{ kip}$$

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

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

$$M_{max} = ((-0.0084395 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.2264 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1956 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2264 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1956 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2264 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1956 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.15625 \text{ kipft}$$

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,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.671 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.308 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.308 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

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;">$Ratio = 0.96556$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</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;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.671 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0032413$</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>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.671 \text{ kip} \rightarrow 8671 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8671 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.64 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.64 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.64 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.85 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.846 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.846 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.12491$$

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

Considering z-direction:

$V_{max} = 0.049115 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.049115 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.00044309$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 47.642 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(47.642 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19088$$

Status: **PASS**
Ratio: **0.190**

Considering z-direction:

$M_{max} = 0.15625 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.15625 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00062599$$

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

REFERENCES	CALCULATIONS	RESULTS
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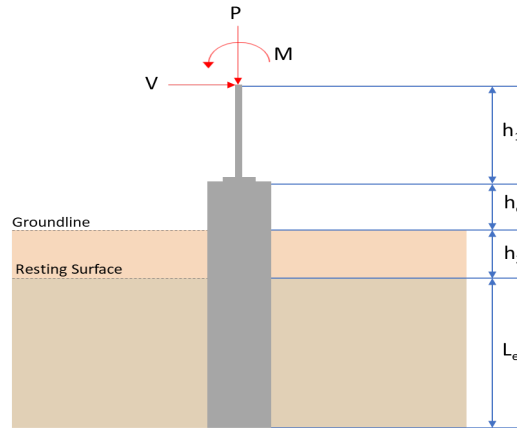
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ 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)	5.690	8.671
V_x (kip)	-3.658	-6.083
V_z (kip)	0.031	0.053
M_x (kipft)	0.101	0.172
M_z (kipft)	43.152	72.630

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,

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

$$H_o = \frac{(-3.658 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.152 \text{ kipft}) + ((-3.658 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8713 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.7863 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.031 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0049363 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.101 \text{ kipft}) + ((0.031 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016083 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1782 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.7863 \text{ ft}), (1.1782 \text{ ft})]$$

$$L_{e,req} = 6.786 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.786 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.936$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.69 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.35563 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35563 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17781$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58248 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8713 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8713 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.8713 \text{ kipft/ft})) + (4 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0089 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8713 \text{ kipft/ft})) + (3 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.8713 \text{ kipft/ft})) + (2 \times (-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.25742 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8713 \text{ kipft/ft})) + ((-0.58248 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0867 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0089 \text{ ft})}{2}$$

$$p_a = 0.37567 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.25742 \text{ kip/ft}^2)}{(0.37567 \text{ kip/ft}^2)}$$

$$Ratio = 0.68524$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0867 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.99924$$

Status: **PASS**
Ratio: **0.690**

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

Considering z-direction:

$H_o = 0.0049363 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.016083 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.016083 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.016083 \text{ kipft/ft})) + (4 \times (0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1942 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.016083 \text{ kipft/ft})) + (3 \times (0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (0.016083 \text{ kipft/ft})) + (2 \times (0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.0035104 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.016083 \text{ kipft/ft})) + ((0.0049363 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.0077569 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1942 \text{ ft})}{2}$$

$$p_a = 0.38957 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0035104 \text{ kip/ft}^2)}{(0.38957 \text{ kip/ft}^2)}$$

$$Ratio = 0.0090111$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

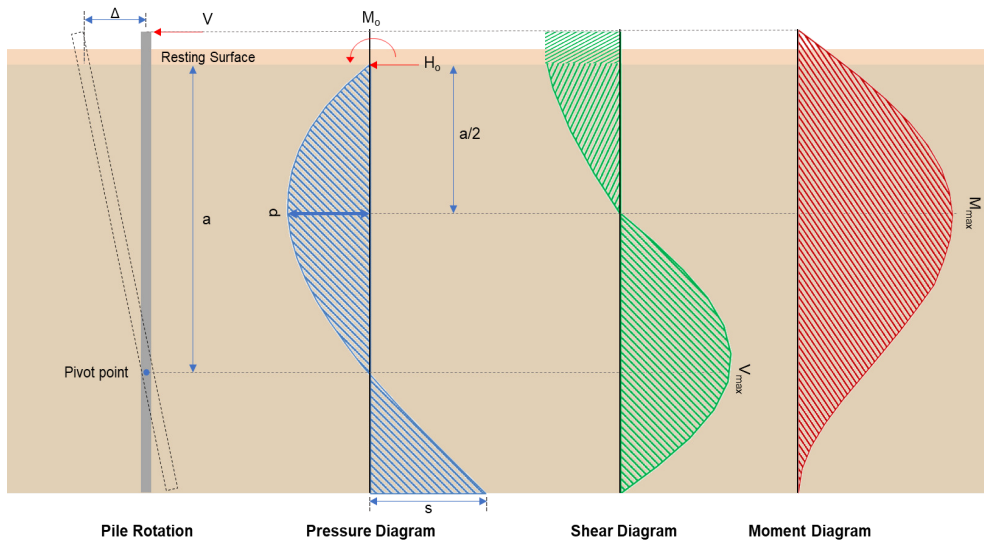
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0077569 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0071328$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.083 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96863 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(72.63 \text{ kipft}) + ((-6.083 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.565 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.565 \text{ kipft/ft})}{(-0.96863 \text{ kip/ft})}$$

$$E = 11.94 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.565 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (11.565 \text{ kipft/ft})) + (4 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = \frac{(-0.96863 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.565 \text{ kipft/ft})) + (4 \times (-0.96863 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0074 \text{ ft}$$

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

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

$$V_{max} = ((-0.96863 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0074 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0074 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.846 \text{ kip}$$

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

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

$$M_{max} = ((-0.96863 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(11.94 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.94 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0074 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 47.643 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.053 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0084395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.172 \text{ kipft}) + ((0.053 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.027389 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.027389 \text{ kipft/ft})}{(0.0084395 \text{ kip/ft})}$$

$$E = 3.2453 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.027389 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0084395 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.027389 \text{ kipft/ft})) + (4 \times (0.0084395 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1948 \text{ ft}$$

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

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

$$V_{max} = ((0.0084395 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2453 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1948 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2453 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1948 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.049269 \text{ kip}$$

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

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

$$M_{max} = ((0.0084395 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.2453 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1948 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2453 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1948 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2453 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1948 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1568 \text{ kipft}$$

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,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.671 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.308 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.308 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

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;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Min[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Min[1.5, (1.5 \times (0.625 \text{ in}))]$</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>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</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;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.671 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0032413$</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>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.671 \text{ kip} \rightarrow 8671 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8671 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.64 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.64 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.64 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.85 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.846 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.846 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.12491$$

Considering z-direction:

$V_{max} = 0.049269 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.049269 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.00044447$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 47.643 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(47.643 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19088$$

Status: **PASS**
Ratio: **0.190**

Considering z-direction:

$M_{max} = 0.1568 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.1568 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00062821$$

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