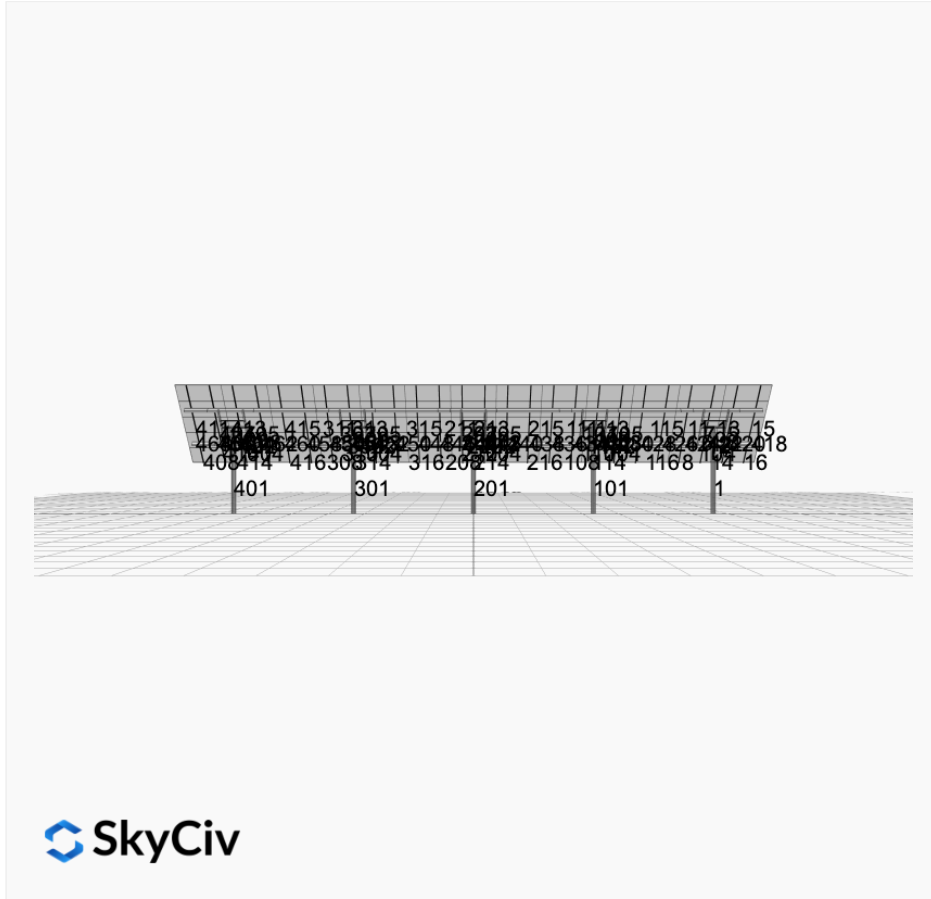


Project Details



Project Name: Woodlawn Beach Solar Canopies 5x13 **Date:** Tue Apr 15 2025
- V1Jb **Number of Modules:** 65
Location: 3580 Lake Shore Rd, Blasdell, NY 14219, USA **Number of Poles:** 5
Unique ID: 5P-19.75-8TOP-HD-45-L-5Hx13W-GK2D **Date Sold:**
Dealer: _____



Array Dimensions N/S	17.33 ft
Array Dimensions E/W	95.55 ft
Winter Tilt Angle	46
Front Edge Clearance	8 ft

MT Solar Bill of Materials (5P-19.75-8TOP-HD-45-L-5Hx13W-GK2D)

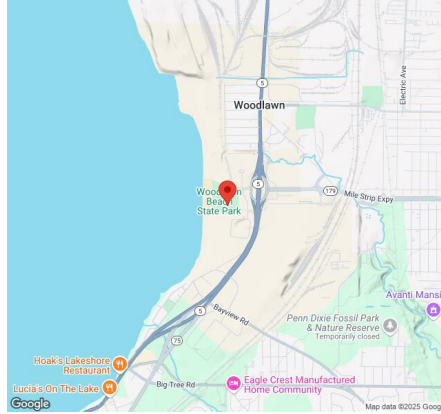
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (208in)	26
Rail Attachment	104

Part	Qty
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: 3580 Lake Shore Rd, Blasdell, NY 14219, USA

Array Specification

Duty Classification:	HD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	46
Front Edge Clearance:	8
Total Array Height at Tilt:	20.47 ft
Total Frame Length:	94.00 ft
Module Info/Notes:	
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	95.55 ft
Rail Length:	208.00 in
Rail Spacing:	3.68 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	14.23 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.25 ft Pile 3: 8.25 ft Pile 4: 8.25 ft Pile 5: 8.00 ft
Foundation Volume:	24.148 y ³

Site Info

Risk Category:	II
Exposure:	D
Soil Classification:	sand
Site Location:	3580 Lake Shore Rd, Blasdell, NY 14219, USA
Wind Speed:	110 mph

Snow Load:

50 psf

Design Disclaimer

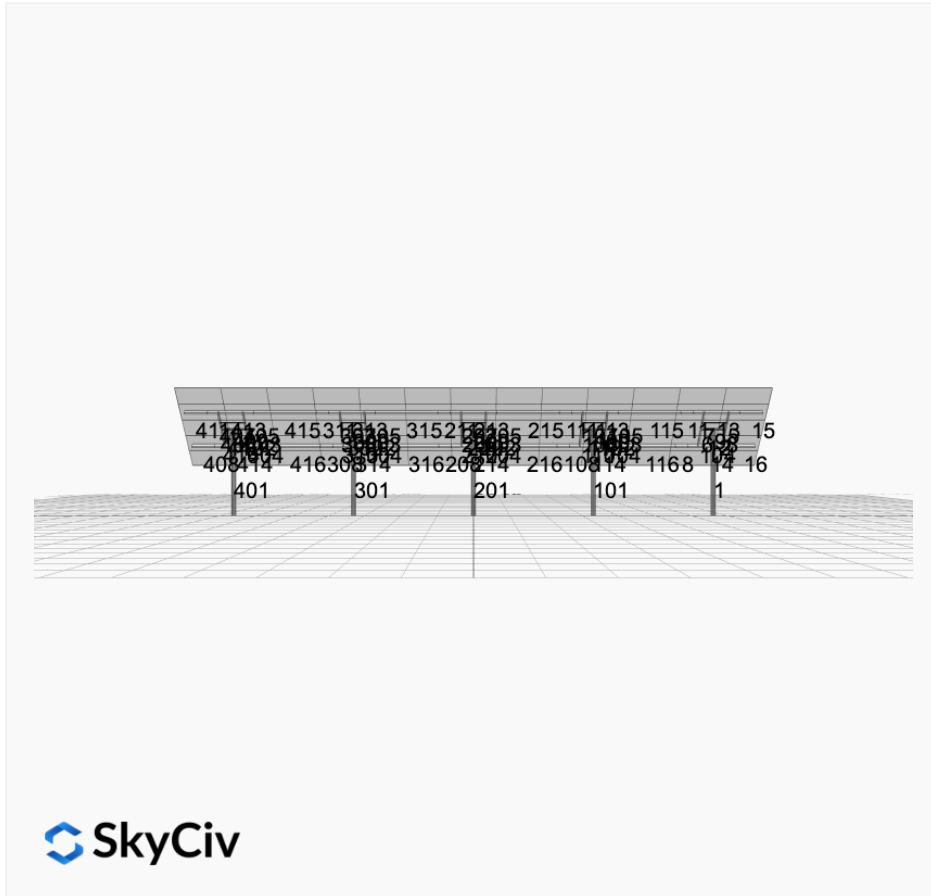
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

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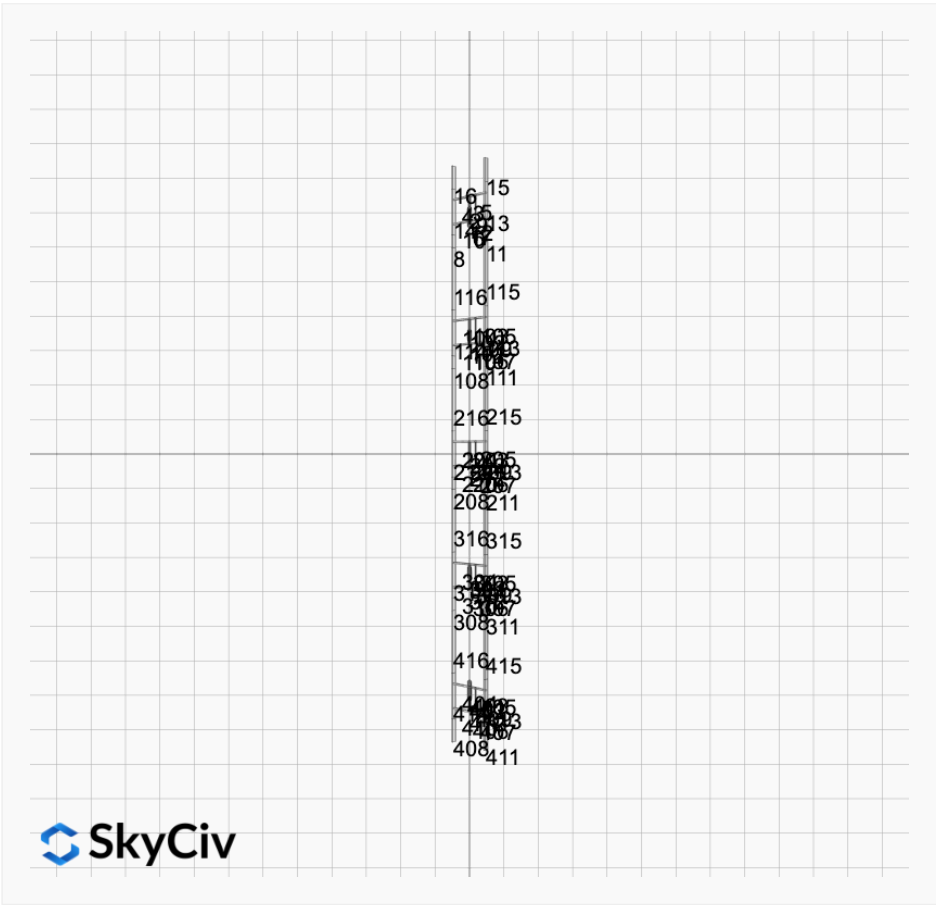
Design Notes:

- 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)

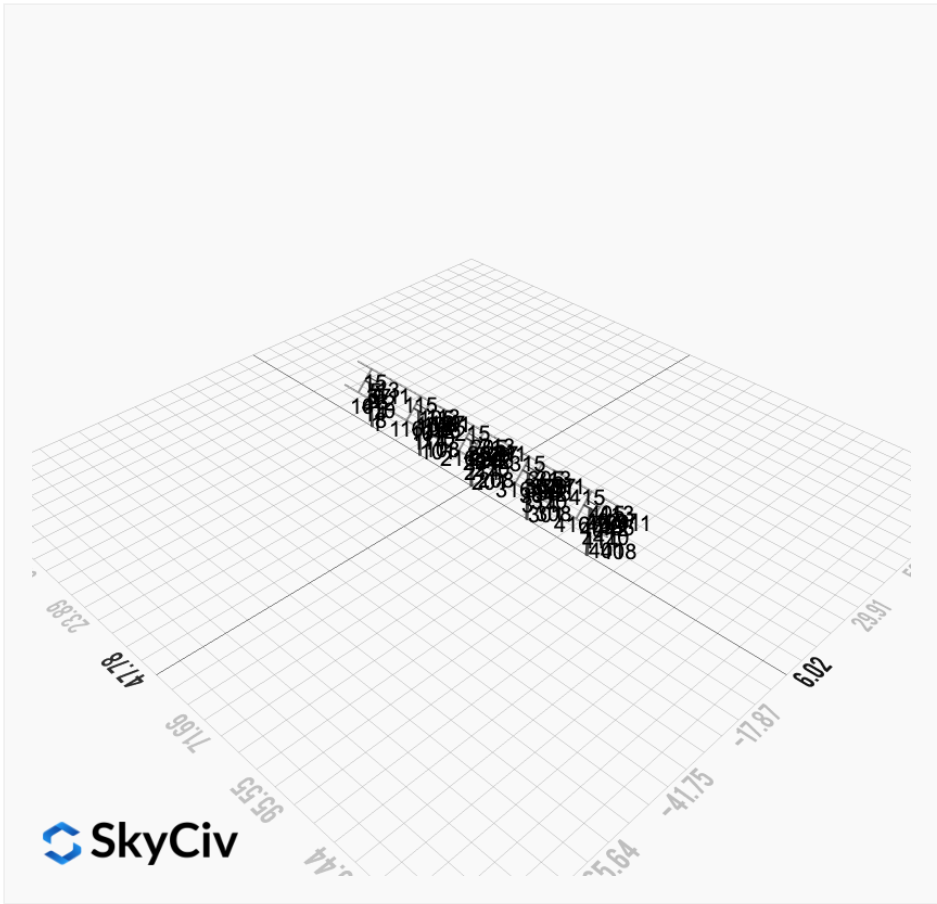




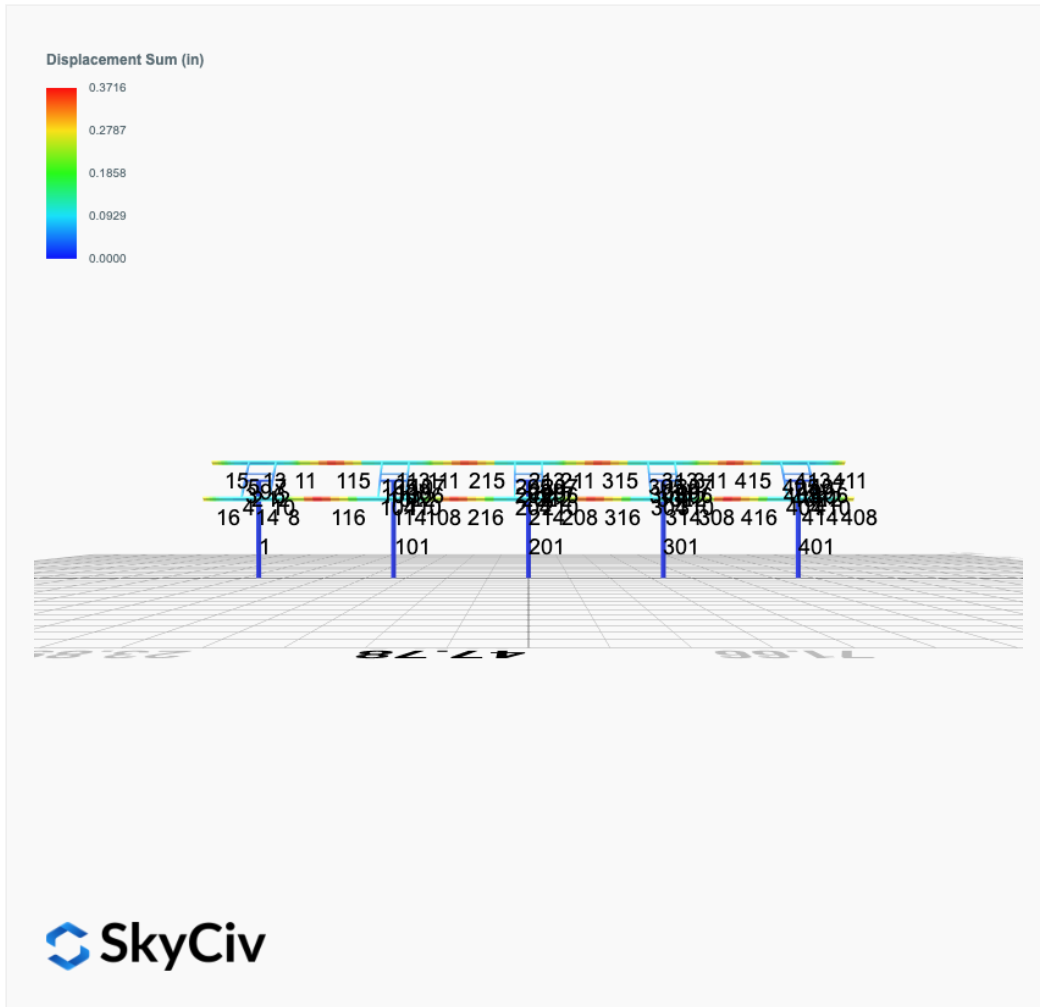
 SkyCiv



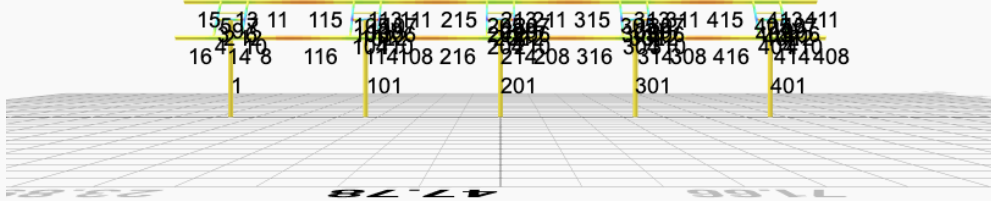
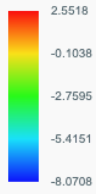
 SkyCiv



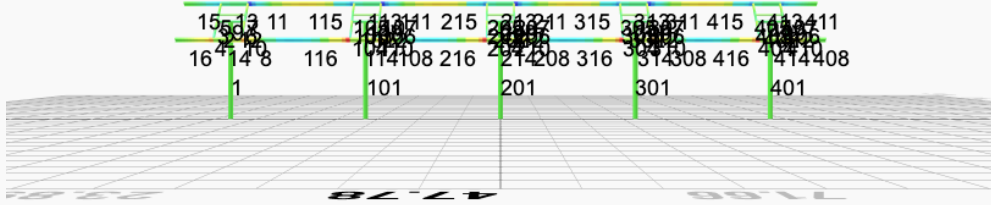
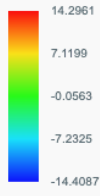
FEM Results (Envelope Worst Case for each member)



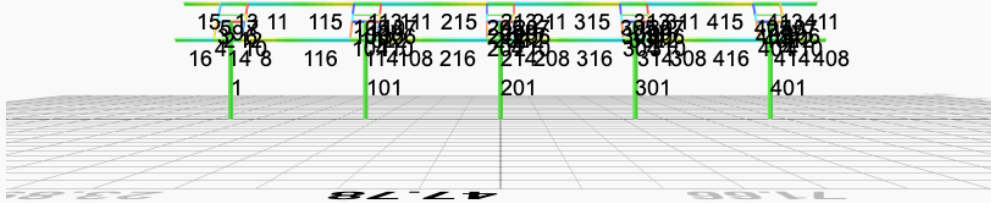
Top Bending Stress Z (ksi)



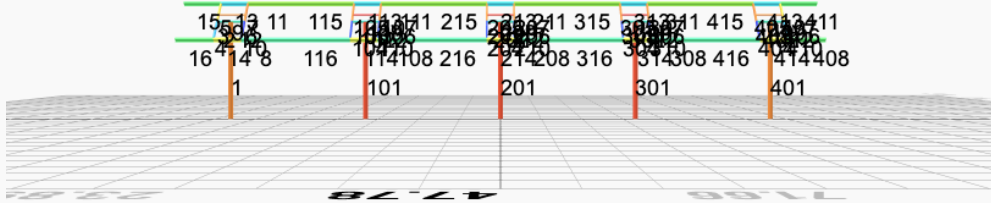
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 2. D + L	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 3. D + (S or Lr or R)	0.0212	5.5916	0.0734	0.3255	-0.0352	-0.2550
ULS: 3. D + (S or Lr or R)	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0178	4.8287	0.0616	0.2732	-0.0295	-0.2115
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 5b. D + 0.7E	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0178	4.8287	0.0616	0.2732	-0.0295	-0.2115
ULS: 8. 0.6D + 0.7E	0.0046	1.5240	0.0157	0.0698	-0.0074	-0.0486
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6905	6.0705	0.1338	0.5624	-0.6372	53.7151
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7007	-0.9886	-0.0770	-0.3109	0.5878	-51.8209
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7557	7.4766	0.1423	0.6078	-0.4982	40.1355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0178	4.8287	0.0616	0.2732	-0.0295	-0.2115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7876	2.1823	-0.0159	-0.0472	0.4206	-39.0165
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0178	4.8287	0.0616	0.2732	-0.0295	-0.2115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7659	5.1879	0.1069	0.4509	-0.4810	40.2661
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7774	-0.1064	-0.0512	-0.2041	0.4378	-38.8859
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0076	2.5399	0.0262	0.1163	-0.0123	-0.0809
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6935	5.0546	0.1233	0.5159	-0.6323	53.7475
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0046	1.5240	0.0157	0.0698	-0.0074	-0.0486
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6976	-2.0045	-0.0875	-0.3574	0.5927	-51.7886
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0046	1.5240	0.0157	0.0698	-0.0074	-0.0486

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.8735
Shear X	-6.1739
Shear Z	0.2368
Moment X	0.9988
Moment Y (Twist)	1.0818
Moment Z	90.9614

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.4766
Shear X	-3.7007
Shear Z	0.1423
Moment X	0.6078
Moment Y (Twist)	0.6372
Moment Z	53.7475

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	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 2. D + L	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 3. D + (S or Lr or R)	-0.0187	6.2932	-0.0022	-0.0105	0.0100	0.2840
ULS: 3. D + (S or Lr or R)	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0157	5.4178	-0.0019	-0.0088	0.0084	0.2407

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 5b. D + 0.7E	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0157	5.4178	-0.0019	-0.0088	0.0084	0.2407
ULS: 8. 0.6D + 0.7E	-0.0040	1.6751	-0.0005	-0.0023	0.0022	0.0665
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1511	6.8282	0.0239	0.0955	-0.1807	60.2301
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.1424	-1.2473	-0.0223	-0.0901	0.1679	-57.5749
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1241	8.4451	0.0166	0.0656	-0.1298	45.3302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0157	5.4178	-0.0019	-0.0088	0.0084	0.2407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.0961	2.3885	-0.0180	-0.0735	0.1316	-43.0235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0157	5.4178	-0.0019	-0.0088	0.0084	0.2407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1150	5.8191	0.0177	0.0707	-0.1346	45.2003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1051	-0.2375	-0.0170	-0.0685	0.1268	-43.1535
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0067	2.7918	-0.0008	-0.0038	0.0036	0.1108
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1485	5.7115	0.0242	0.0970	-0.1822	60.1858
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0040	1.6751	-0.0005	-0.0023	0.0022	0.0665
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.1451	-2.3640	-0.0220	-0.0886	0.1664	-57.6192
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0040	1.6751	-0.0005	-0.0023	0.0022	0.0665

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	12.3148
Shear X	-6.9181
Shear Z	0.0414
Moment X	0.1656
Moment Y (Twist)	0.3126
Moment Z	102.1614

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.4451
Shear X	-4.1511
Shear Z	0.0242
Moment X	0.0970
Moment Y (Twist)	0.1822
Moment Z	60.2301

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	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 2. D + L	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 3. D + (S or Lr or R)	-0.0049	6.2733	0.0000	-0.0000	0.0000	0.1296
ULS: 3. D + (S or Lr or R)	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0041	5.4012	0.0000	-0.0000	0.0000	0.1111
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 5b. D + 0.7E	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0041	5.4012	0.0000	-0.0000	0.0000	0.1111
ULS: 8. 0.6D + 0.7E	-0.0011	1.6708	0.0000	0.0000	0.0000	0.0333
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1949	6.8467	0.0000	-0.0000	0.0000	61.0836
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.1919	-1.2762	0.0000	-0.0000	0.0000	-58.4169
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554

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	-3.1490	8.4477	0.0000	-0.0000	0.0000	45.8822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0041	5.4012	0.0000	-0.0000	0.0000	0.1111
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1411	2.3555	0.0000	-0.0000	0.0000	-43.7432
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0041	5.4012	0.0000	-0.0000	0.0000	0.1111
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1466	5.8312	0.0000	-0.0000	0.0000	45.8266
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1435	-0.2610	0.0000	-0.0000	0.0000	-43.7988
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0018	2.7847	0.0000	0.0000	0.0000	0.0554
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1942	5.7328	0.0000	-0.0000	0.0000	61.0615
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0011	1.6708	0.0000	0.0000	0.0000	0.0333
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.1926	-2.3901	0.0000	-0.0000	0.0000	-58.4390
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0011	1.6708	0.0000	0.0000	0.0000	0.0333

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	12.3095
Shear X	-6.9922
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0003
Moment Z	103.6400

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.4477
Shear X	-4.1949
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	61.0836

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	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 2. D + L	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 3. D + (S or Lr or R)	-0.0187	6.2932	0.0022	0.0104	-0.0100	0.2840
ULS: 3. D + (S or Lr or R)	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0157	5.4178	0.0019	0.0088	-0.0084	0.2407
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 5b. D + 0.7E	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0157	5.4178	0.0019	0.0088	-0.0084	0.2407
ULS: 8. 0.6D + 0.7E	-0.0040	1.6751	0.0005	0.0023	-0.0022	0.0665
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1511	6.8282	-0.0239	-0.0955	0.1807	60.2301
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.1424	-1.2473	0.0223	0.0901	-0.1678	-57.5749
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1241	8.4451	-0.0166	-0.0657	0.1299	45.3302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0157	5.4178	0.0019	0.0088	-0.0084	0.2407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.0961	2.3885	0.0180	0.0735	-0.1316	-43.0235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0157	5.4178	0.0019	0.0088	-0.0084	0.2407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1150	5.8191	-0.0177	-0.0707	0.1347	45.2003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1051	-0.2375	0.0170	0.0685	-0.1268	-43.1535
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0067	2.7918	0.0008	0.0038	-0.0036	0.1108

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1485	5.7115	-0.0242	-0.0970	0.1822	60.1858
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0040	1.6751	0.0005	0.0023	-0.0022	0.0665
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.1451	-2.3640	0.0220	0.0886	-0.1664	-57.6192
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0040	1.6751	0.0005	0.0023	-0.0022	0.0665

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	12.3148
Shear X	-6.9182
Shear Z	-0.0414
Moment X	-0.1658
Moment Y (Twist)	0.3128
Moment Z	102.1617

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.4451
Shear X	-4.1511
Shear Z	-0.0242
Moment X	-0.0970
Moment Y (Twist)	0.1822
Moment Z	60.2301

Reaction Forces for Foundation 5 (Node ID#401), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 2. D + L	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 3. D + (S or Lr or R)	0.0212	5.5916	-0.0734	-0.3256	0.0353	-0.2549
ULS: 3. D + (S or Lr or R)	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0178	4.8287	-0.0616	-0.2732	0.0295	-0.2114
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 5b. D + 0.7E	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0178	4.8287	-0.0616	-0.2732	0.0295	-0.2114
ULS: 8. 0.6D + 0.7E	0.0046	1.5240	-0.0157	-0.0698	0.0074	-0.0486
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6905	6.0705	-0.1338	-0.5625	0.6372	53.7151
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7007	-0.9886	0.0770	0.3109	-0.5878	-51.8209
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7557	7.4766	-0.1423	-0.6079	0.4982	40.1356
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0178	4.8287	-0.0616	-0.2732	0.0295	-0.2114
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7876	2.1823	0.0159	0.0471	-0.4205	-39.0164
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0178	4.8287	-0.0616	-0.2732	0.0295	-0.2114
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7659	5.1879	-0.1069	-0.4509	0.4810	40.2661
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7774	-0.1064	0.0512	0.2041	-0.4378	-38.8859
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0076	2.5399	-0.0262	-0.1163	0.0123	-0.0809
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6935	5.0546	-0.1233	-0.5159	0.6323	53.7475
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0046	1.5240	-0.0157	-0.0698	0.0074	-0.0486
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6976	-2.0045	0.0875	0.3574	-0.5927	-51.7886
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0046	1.5240	-0.0157	-0.0698	0.0074	-0.0486

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.

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	10.8734
Shear X	-6.1739
Shear Z	-0.2368
Moment X	-0.9990
Moment Y (Twist)	1.0819
Moment Z	90.9625

Result	Value (kip, kip-ft)
Axial	7.4766
Shear X	-3.7007
Shear Z	-0.1423
Moment X	-0.6079
Moment Y (Twist)	0.6372
Moment Z	53.7475

Project Details

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

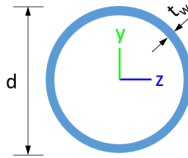


Design Input Information

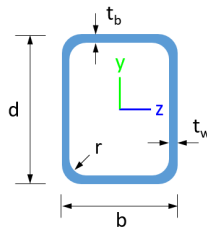
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

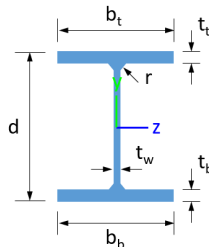
Section Dimensions



ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



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



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

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

104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.93	6.12	40.24	43.62
114	133.20	85.85	23.89	6.12	40.24	43.62
115	133.20	69.16	17.16	6.12	40.24	43.62
116	133.20	69.16	17.62	6.12	40.24	43.62
201	574.32	185.61	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.55	6.12	40.24	43.62
214	133.20	85.85	23.75	6.12	40.24	43.62
215	133.20	69.16	17.71	6.12	40.24	43.62
216	133.20	69.16	17.88	6.12	40.24	43.62
301	574.32	185.61	123.94	123.94	172.30	172.30
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	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.93	6.12	40.24	43.62
314	133.20	85.85	23.89	6.12	40.24	43.62
315	133.20	69.16	17.49	6.12	40.24	43.62
316	133.20	69.16	17.72	6.12	40.24	43.62
401	574.32	185.61	123.94	123.94	172.30	172.30
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28

407	110.10	114.23	13.79	11.10	42.00	23.20
408	133.20	52.83	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	52.83	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	24.61	6.12	40.24	43.62
414	133.20	85.85	24.88	6.12	40.24	43.62
415	133.20	69.16	17.42	6.12	40.24	43.62
416	133.20	69.16	17.37	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	0.059	0.734	0.019	0.036	0.001	0.770	#13	0.623	Not Required	Pass
2	0.003	0.316	0.248	0.076	0.048	0.562	#13	0.035	Not Required	Pass
3	0.010	0.608	0.049	0.060	0.005	0.639	#13	0.045	Not Required	Pass
4	0.009	0.608	0.155	0.061	0.034	0.680	#13	0.080	Not Required	Pass
5	0.009	0.378	0.150	0.060	0.038	0.394	#13	0.074	Not Required	Pass
6	0.013	0.726	0.089	0.073	0.017	0.778	#13	0.045	Not Required	Pass
7	0.013	0.451	0.210	0.072	0.053	0.477	#13	0.074	Not Required	Pass
8	0.002	0.085	0.188	0.047	0.021	0.189	#23	0.095	Not Required	Pass
9	0.014	0.058	0.071	0.003	0.002	0.132	#13	0.204	Not Required	Pass
10	0.013	0.704	0.205	0.070	0.044	0.751	#13	0.080	Not Required	Pass
11	0.003	0.077	0.191	0.049	0.020	0.197	#23	0.095	Not Required	Pass
12	0.003	0.421	0.297	0.094	0.057	0.719	#13	0.053	Not Required	Pass
13	0.007	0.221	0.484	0.063	0.026	0.606	#21	0.286	Not Required	Pass
14	0.009	0.215	0.479	0.061	0.026	0.586	#21	0.190	Not Required	Pass
15	0.000	0.068	0.166	0.030	0.012	0.222	#21	Not Required	Not Required	Pass
16	0.000	0.068	0.166	0.030	0.012	0.222	#21	Not Required	Not Required	Pass
101	0.066	0.824	0.003	0.040	0.000	0.858	#13	0.623	Not Required	Pass
102	0.004	0.407	0.300	0.097	0.057	0.708	#13	0.035	Not Required	Pass
103	0.013	0.735	0.078	0.073	0.012	0.781	#13	0.045	Not Required	Pass
104	0.013	0.748	0.205	0.075	0.044	0.832	#13	0.080	Not Required	Pass
105	0.013	0.456	0.212	0.073	0.053	0.485	#13	0.074	Not Required	Pass
106	0.013	0.764	0.078	0.076	0.012	0.805	#13	0.045	Not Required	Pass
107	0.013	0.475	0.205	0.076	0.051	0.504	#13	0.074	Not Required	Pass
108	0.003	0.064	0.180	0.048	0.020	0.208	#21	0.095	Not Required	Pass
109	0.016	0.056	0.060	0.001	0.000	0.121	#13	0.204	Not Required	Pass
110	0.013	0.761	0.198	0.076	0.042	0.828	#13	0.080	Not Required	Pass
111	0.003	0.072	0.184	0.048	0.020	0.208	#21	0.095	Not Required	Pass
112	0.004	0.426	0.314	0.098	0.060	0.742	#13	0.035	Not Required	Pass
113	0.007	0.213	0.490	0.062	0.026	0.648	#21	0.286	Not Required	Pass
114	0.011	0.236	0.486	0.063	0.026	0.659	#21	0.286	Not Required	Pass
115	0.005	0.303	0.262	0.048	0.021	0.511	#21	0.473	Not Required	Pass
116	0.003	0.296	0.264	0.049	0.021	0.506	#21	0.473	Not Required	Pass
201	0.066	0.836	0.000	0.041	0.000	0.868	#13	0.623	Not Required	Pass
202	0.004	0.419	0.311	0.097	0.059	0.731	#13	0.035	Not Required	Pass
203	0.013	0.760	0.077	0.076	0.012	0.804	#13	0.045	Not Required	Pass
204	0.013	0.754	0.197	0.075	0.042	0.827	#13	0.080	Not Required	Pass
205	0.013	0.472	0.204	0.075	0.051	0.499	#13	0.074	Not Required	Pass

206	0.013	0.759	0.077	0.076	0.012	0.804	#13	0.045	Not Required	Pass
207	0.013	0.472	0.205	0.075	0.051	0.499	#13	0.074	Not Required	Pass
208	0.003	0.061	0.180	0.049	0.020	0.213	#21	0.095	Not Required	Pass
209	0.016	0.051	0.061	0.001	0.000	0.116	#13	0.204	Not Required	Pass
210	0.013	0.754	0.197	0.075	0.042	0.827	#13	0.080	Not Required	Pass
211	0.003	0.067	0.183	0.049	0.020	0.216	#21	0.095	Not Required	Pass
212	0.004	0.419	0.311	0.097	0.059	0.731	#13	0.035	Not Required	Pass
213	0.007	0.234	0.473	0.062	0.026	0.653	#21	0.286	Not Required	Pass
214	0.011	0.246	0.469	0.062	0.026	0.651	#21	0.286	Not Required	Pass
215	0.005	0.276	0.263	0.049	0.020	0.486	#21	0.473	Not Required	Pass
216	0.004	0.262	0.263	0.048	0.020	0.478	#21	0.473	Not Required	Pass
301	0.066	0.824	0.003	0.040	0.000	0.858	#13	0.623	Not Required	Pass
302	0.004	0.426	0.314	0.098	0.060	0.742	#13	0.035	Not Required	Pass
303	0.013	0.764	0.078	0.076	0.012	0.805	#13	0.045	Not Required	Pass
304	0.013	0.761	0.198	0.076	0.042	0.828	#13	0.080	Not Required	Pass
305	0.013	0.475	0.205	0.076	0.051	0.504	#13	0.074	Not Required	Pass
306	0.013	0.735	0.078	0.073	0.012	0.781	#13	0.045	Not Required	Pass
307	0.013	0.456	0.212	0.073	0.053	0.485	#13	0.074	Not Required	Pass
308	0.002	0.074	0.193	0.049	0.021	0.220	#21	0.095	Not Required	Pass
309	0.016	0.056	0.060	0.001	0.000	0.121	#13	0.204	Not Required	Pass
310	0.013	0.748	0.205	0.075	0.044	0.832	#13	0.080	Not Required	Pass
311	0.003	0.085	0.196	0.048	0.021	0.215	#21	0.095	Not Required	Pass
312	0.004	0.407	0.300	0.097	0.057	0.708	#13	0.035	Not Required	Pass
313	0.007	0.213	0.490	0.062	0.026	0.648	#21	0.286	Not Required	Pass
314	0.011	0.236	0.486	0.063	0.026	0.659	#21	0.286	Not Required	Pass
315	0.005	0.277	0.263	0.048	0.020	0.487	#21	0.473	Not Required	Pass
316	0.004	0.263	0.263	0.048	0.020	0.479	#21	0.473	Not Required	Pass
401	0.059	0.734	0.019	0.036	0.001	0.770	#13	0.623	Not Required	Pass
402	0.003	0.421	0.297	0.094	0.057	0.719	#13	0.053	Not Required	Pass
403	0.013	0.726	0.089	0.073	0.017	0.778	#13	0.045	Not Required	Pass
404	0.013	0.704	0.205	0.070	0.044	0.751	#13	0.080	Not Required	Pass
405	0.013	0.451	0.210	0.072	0.053	0.477	#13	0.074	Not Required	Pass
406	0.010	0.609	0.049	0.060	0.005	0.639	#13	0.045	Not Required	Pass
407	0.009	0.378	0.150	0.060	0.038	0.394	#13	0.074	Not Required	Pass
408	0.000	0.068	0.166	0.030	0.012	0.222	#21	Not Required	Not Required	Pass
409	0.014	0.058	0.071	0.003	0.002	0.132	#13	0.204	Not Required	Pass
410	0.009	0.608	0.155	0.061	0.034	0.680	#13	0.080	Not Required	Pass
411	0.000	0.068	0.166	0.030	0.012	0.222	#21	Not Required	Not Required	Pass
412	0.003	0.316	0.248	0.076	0.048	0.562	#13	0.035	Not Required	Pass
413	0.007	0.221	0.484	0.063	0.026	0.606	#21	0.190	Not Required	Pass
414	0.009	0.215	0.479	0.061	0.026	0.586	#21	0.286	Not Required	Pass
415	0.005	0.300	0.263	0.049	0.020	0.510	#21	0.473	Not Required	Pass
416	0.003	0.300	0.262	0.047	0.021	0.509	#21	0.473	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength

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

REFERENCES	CALCULATIONS	RESULTS
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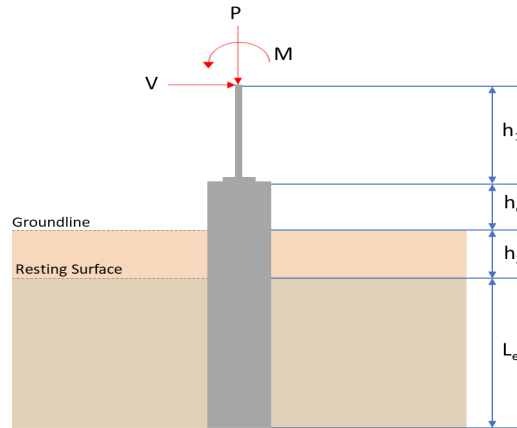
SkyCiv Foundation Design

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

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.477	10.873
V_x (kip)	-3.701	-6.174
V_z (kip)	0.142	0.237
M_x (kipft)	0.608	0.999
M_z (kipft)	53.747	90.961

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,

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

$$H_o = \frac{(-3.701 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58933 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.747 \text{ kipft}) + ((-3.701 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.5584 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.4882 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.022611 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.608 \text{ kipft}) + ((0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.096815 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2062 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.4882 \text{ ft}), (2.2062 \text{ ft})]$$

$$L_{e,req} = 7.488 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.488 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.936$$

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

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

$$q = \frac{(7.477 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46731 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46731 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23366$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58933 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.5584 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.5584 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.5584 \text{ kipft/ft})) + (4 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5124 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.5584 \text{ kipft/ft})) + (3 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.5584 \text{ kipft/ft})) + (2 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.29113 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.5584 \text{ kipft/ft})) + ((-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1627 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5124 \text{ ft})}{2}$$

$$p_a = 0.41343 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29113 \text{ kip/ft}^2)}{(0.41343 \text{ kip/ft}^2)}$$

$$Ratio = 0.70418$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1627 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.96892$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = 0.022611 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.096815 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.096815 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.022611 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.096815 \text{ kipft/ft})) + (4 \times (0.022611 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7031 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.096815 \text{ kipft/ft})) + (3 \times (0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.096815 \text{ kipft/ft})) + (2 \times (0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.015538 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.096815 \text{ kipft/ft})) + ((0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.035111 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7031 \text{ ft})}{2}$$

$$p_a = 0.42773 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.015538 \text{ kip/ft}^2)}{(0.42773 \text{ kip/ft}^2)}$$

$$Ratio = 0.036326$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

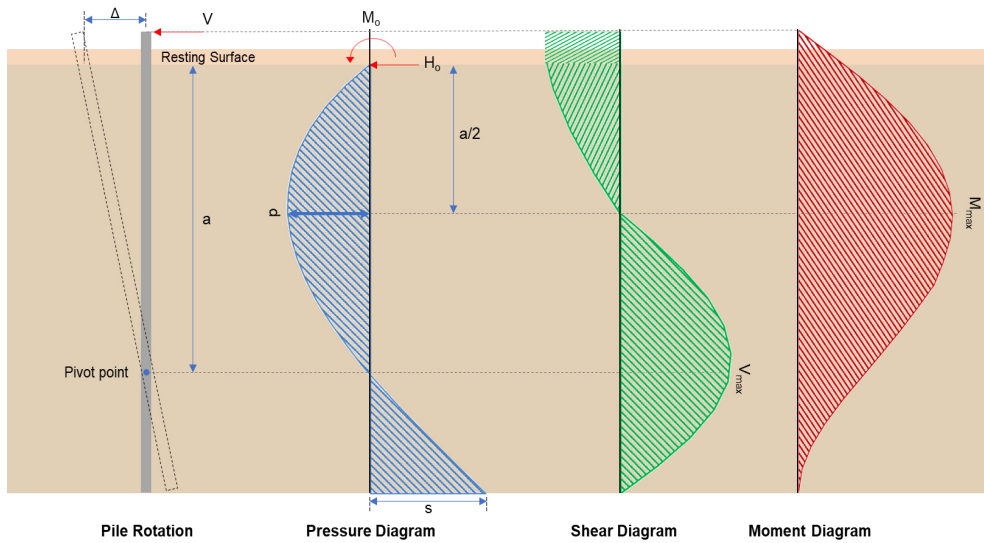
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.035111 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.02926$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.174 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.98312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(90.961 \text{ kipft}) + ((-6.174 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.484 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.484 \text{ kipft/ft})}{(-0.98312 \text{ kip/ft})}$$

$$E = 14.733 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.484 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.484 \text{ kipft/ft})) + (4 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(-0.98312 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (14.484 \text{ kip/ft})) + (4 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5105 \text{ ft}$$

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

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

$$V_{max} = ((-0.98312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5105 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5105 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.41 \text{ kip}$$

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

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

$$M_{max} = ((-0.98312 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(14.733 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 58.78 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.237 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.037739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.999 \text{ kipft}) + ((0.237 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15908 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15908 \text{ kipft/ft})}{(0.037739 \text{ kip/ft})}$$

$$E = 4.2152 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15908 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.037739 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.15908 \text{ kipft/ft})) + (4 \times (0.037739 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7057 \text{ ft}$$

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

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

$$V_{max} = ((0.037739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.24124 \text{ kip}$$

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

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

$$M_{max} = ((0.037739 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2152 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85717 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.873 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.235 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.235 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(10.873 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0040644$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.873 \text{ kip} \rightarrow 10873 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10873 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.94 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.94 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.94 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.94 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.04 \text{ kip}$$

Considering x-direction:

$V_{max} = 15.41 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(15.41 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.13878$$

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

Considering z-direction:

$V_{max} = 0.24124 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.24124 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.0021726$$

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:

$M_{max} = 58.78 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(58.78 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2355$$

Status: **PASS**
Ratio: **0.240**

Considering z-direction:

$M_{max} = 0.85717 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.85717 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0034342$$

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

REFERENCES	CALCULATIONS	RESULTS
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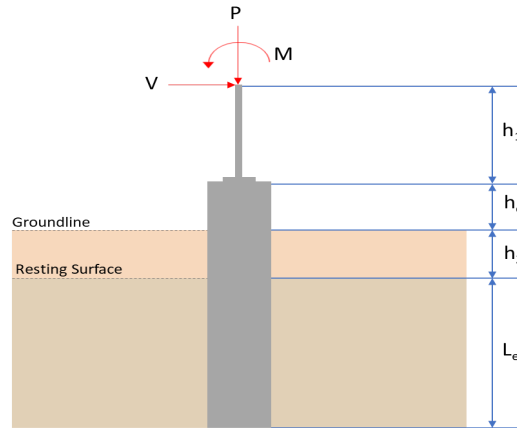
SkyCiv Foundation Design

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

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.477	10.873
V_x (kip)	-3.701	-6.174
V_z (kip)	-0.142	-0.237
M_x (kipft)	-0.608	-0.999
M_z (kipft)	53.747	90.962

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,

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

$$H_o = \frac{(-3.701 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58933 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.747 \text{ kipft}) + ((-3.701 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.5584 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.4882 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.022611 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.608 \text{ kipft}) + ((-0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.096815 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7512 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.4882 \text{ ft}), (1.7512 \text{ ft})]$$

$$L_{e,req} = 7.488 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.488 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.936$$

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

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

$$q = \frac{(7.477 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46731 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46731 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23366$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58933 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.5584 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.5584 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.5584 \text{ kipft/ft})) + (4 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5124 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.5584 \text{ kipft/ft})) + (3 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.5584 \text{ kipft/ft})) + (2 \times (-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.29113 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.5584 \text{ kipft/ft})) + ((-0.58933 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1627 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5124 \text{ ft})}{2}$$

$$p_a = 0.41343 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29113 \text{ kip/ft}^2)}{(0.41343 \text{ kip/ft}^2)}$$

$$Ratio = 0.70418$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1627 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.96892$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = -0.022611 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.096815 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.096815 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.022611 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.096815 \text{ kipft/ft})) + (4 \times (-0.022611 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7031 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.096815 \text{ kipft/ft})) + (3 \times (-0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 [(3 \times (0.096815 \text{ kipft/ft})) + (2 \times (-0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.0039677 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.096815 \text{ kipft/ft})) + ((-0.022611 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.0011943 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7031 \text{ ft})}{2}$$

$$p_a = 0.42773 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0039677 \text{ kip/ft}^2)}{(0.42773 \text{ kip/ft}^2)}$$

$$Ratio = -0.0092762$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

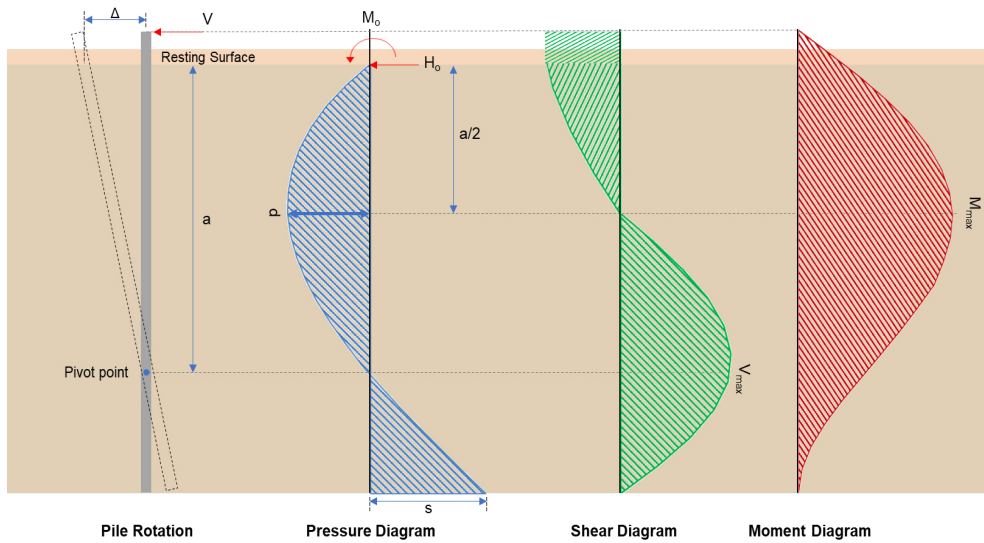
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0011943 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.00099522$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.174 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.98312 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(90.962 \text{ kipft}) + ((-6.174 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.484 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.484 \text{ kipft/ft})}{(-0.98312 \text{ kip/ft})}$$

$$E = 14.733 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.484 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.484 \text{ kipft/ft})) + (4 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(-0.98312 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (14.484 \text{ kipft/ft})) + (4 \times (-0.98312 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5105 \text{ ft}$$

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

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

$$V_{max} = ((-0.98312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5105 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5105 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.41 \text{ kip}$$

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

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

$$M_{max} = ((-0.98312 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(14.733 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.733 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5105 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 58.78 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.237 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.037739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.999 \text{ kipft}) + ((-0.237 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15908 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15908 \text{ kipft/ft})}{(-0.037739 \text{ kip/ft})}$$

$$E = 4.2152 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15908 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.037739 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.15908 \text{ kipft/ft})) + (4 \times (-0.037739 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7057 \text{ ft}$$

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

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

$$V_{max} = ((-0.037739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.24124 \text{ kip}$$

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

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

$$M_{max} = ((-0.037739 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2152 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2152 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85717 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.873 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.235 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.235 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.873 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0040644$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.873 \text{ kip} \rightarrow 10873 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10873 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.94 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.94 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.94 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.94 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.04 \text{ kip}$$

Considering x-direction:

V_{max} = 15.41 kip - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(15.41 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.13878$$

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

Considering z-direction:

$V_{max} = 0.24124 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.24124 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.0021726$$

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:

$M_{max} = 58.78 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(58.78 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2355$$

Status: **PASS**
Ratio: **0.240**

Considering z-direction:

$M_{max} = 0.85717 \text{ kipft}$ - Maximum moment in the z-direction,

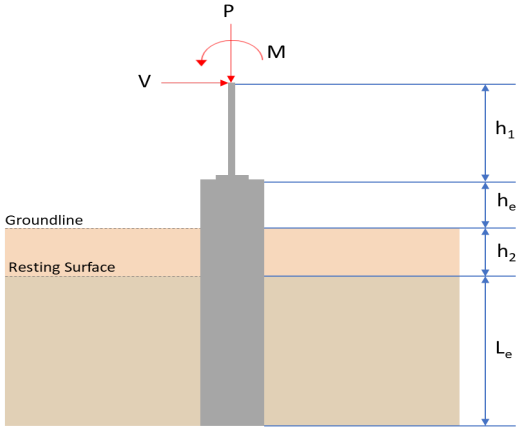
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.85717 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0034342$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 8.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="652 1290 943 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.445</td> <td>12.315</td> </tr> <tr> <td>V_x (kip)</td> <td>-4.151</td> <td>-6.918</td> </tr> <tr> <td>V_z (kip)</td> <td>0.024</td> <td>0.041</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.097</td> <td>0.166</td> </tr> <tr> <td>M_z (kipft)</td> <td>60.230</td> <td>102.161</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength.</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	8.445	12.315	V_x (kip)	-4.151	-6.918	V_z (kip)	0.024	0.041	M_x (kipft)	0.097	0.166	M_z (kipft)	60.230	102.161	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
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M_z (kipft)	60.230	102.161																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-4.151 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.66099 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(60.23 \text{ kipft}) + ((-4.151 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.5908 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.7245 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.097 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015446 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1443 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.7245 \text{ ft}), (1.1443 \text{ ft})]$$

$$L_{e,req} = 7.725 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.725 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.93636$$

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

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

$$q = \frac{(8.445 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.52781 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.52781 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26391$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.66099 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.5908 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.5908 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.5908 \text{ kipft/ft})) + (4 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.689 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.5908 \text{ kipft/ft})) + (3 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.5908 \text{ kipft/ft})) + (2 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.29862 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.5908 \text{ kipft/ft})) + ((-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.2102 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.689 \text{ ft})}{2}$$

$$p_a = 0.42667 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29862 \text{ kip/ft}^2)}{(0.42667 \text{ kip/ft}^2)}$$

$$Ratio = 0.69987$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2102 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.97795$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = 0.0038217 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.015446 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015446 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.015446 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8963 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.015446 \text{ kipft/ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 [(3 \times (0.015446 \text{ kipft/ft})) + (2 \times (0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.002463 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.015446 \text{ kipft/ft})) + ((0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.0055026 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8963 \text{ ft})}{2}$$

$$p_a = 0.44222 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.002463 \text{ kip/ft}^2)}{(0.44222 \text{ kip/ft}^2)}$$

$$Ratio = 0.0055695$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

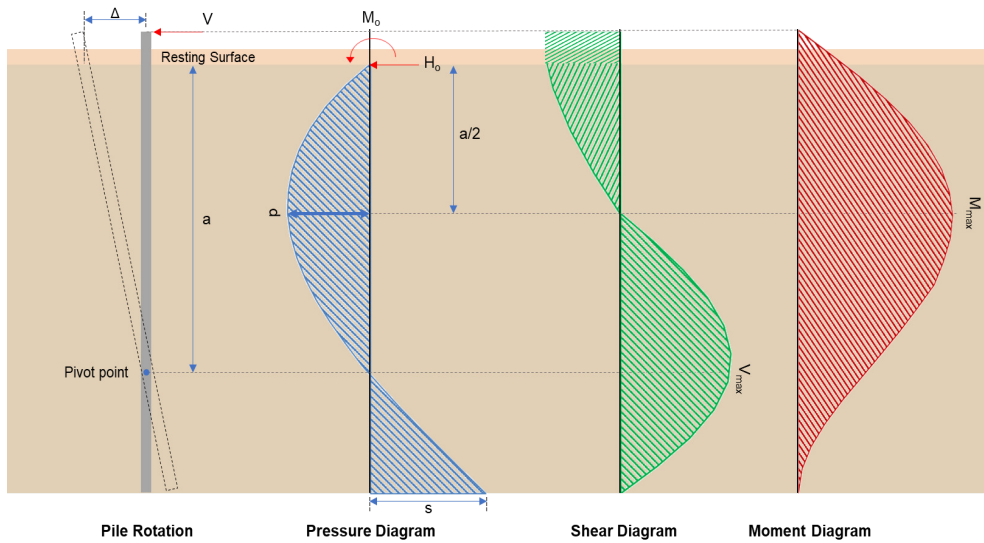
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0055026 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0044466$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.918 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1016 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(102.16 \text{ kipft}) + ((-6.918 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.268 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.268 \text{ kipft/ft})}{(-1.1016 \text{ kip/ft})}$$

$$E = 14.767 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.268 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.1016 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 16.268 \text{ kipft/ft}) + (4 \times (-1.1016 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (16.268 \text{ kipft/ft})) + (4 \times (-1.1016 \text{ kip/ft}) \times (8.25 \text{ ft}))}{}$$

$$a = 5.6866 \text{ ft}$$

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

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

$$V_{max} = ((-1.1016 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.767 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6866 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.767 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6866 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.864 \text{ kip}$$

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

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

$$M_{max} = ((-1.1016 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(14.767 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.767 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.767 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 66.26 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.166 \text{ kipft}) + ((0.041 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.026433 \text{ kipft/ft})}{(0.0065287 \text{ kip/ft})}$$

$$E = 4.0488 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026433 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0065287 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.026433 \text{ kipft/ft})) + (4 \times (0.0065287 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.896 \text{ ft}$$

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

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

$$V_{max} = ((0.0065287 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.896 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.896 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.040082 \text{ kip}$$

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

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

$$M_{max} = ((0.0065287 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(4.0488 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.14613 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.315 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.187 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.187 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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;">$Ratio = \frac{(12.315 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0046034$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.315 \text{ kip} \rightarrow 12315 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12315 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.13 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.13 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.13 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.13 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.16 \text{ kip}$$

Considering x-direction:

V_{max} = 16.864 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.864 \text{ kip})}{(111.16 \text{ kip})}$$

$$Ratio = 0.1517$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.040082 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.040082 \text{ kip})}{(111.16 \text{ kip})}$$

$$Ratio = 0.00036057$$

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

$\phi M_{n,2}$

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

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

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

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:

$M_{max} = 66.26 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(66.26 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.26546$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.14613 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.14613 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00058545$$

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

REFERENCES	CALCULATIONS	RESULTS
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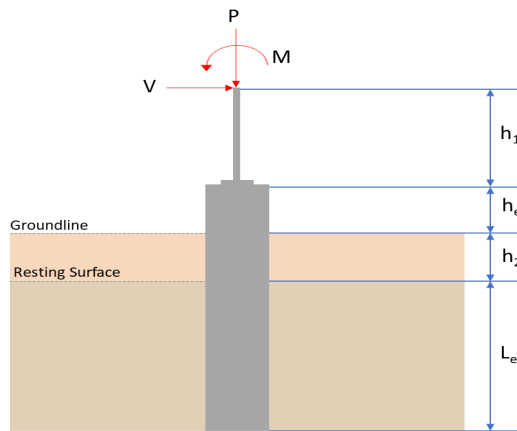
SkyCiv Foundation Design

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

$L = 8.25$ 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)	8.448	12.309
V_x (kip)	-4.195	-6.992
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	61.084	103.640

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,

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

$$H_o = \frac{(-4.195 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66799 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.084 \text{ kipft}) + ((-4.195 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.7268 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.7593 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.7593 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.759 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.759 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.94048$$

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

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

$$q = \frac{(8.448 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.528 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.528 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.264$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.66799 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.7268 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7268 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.66799 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.7268 \text{ kipft/ft})) + (4 \times (-0.66799 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6885 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (9.7268 \text{ kipft/ft})) + (3 \times (-0.66799 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 [(3 \times (9.7268 \text{ kipft/ft})) + (2 \times (-0.66799 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.30379 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (9.7268 \text{ kipft/ft})) + ((-0.66799 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.2291 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6885 \text{ ft})}{2}$$

$$p_a = 0.42664 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30379 \text{ kip/ft}^2)}{(0.42664 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.71205$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

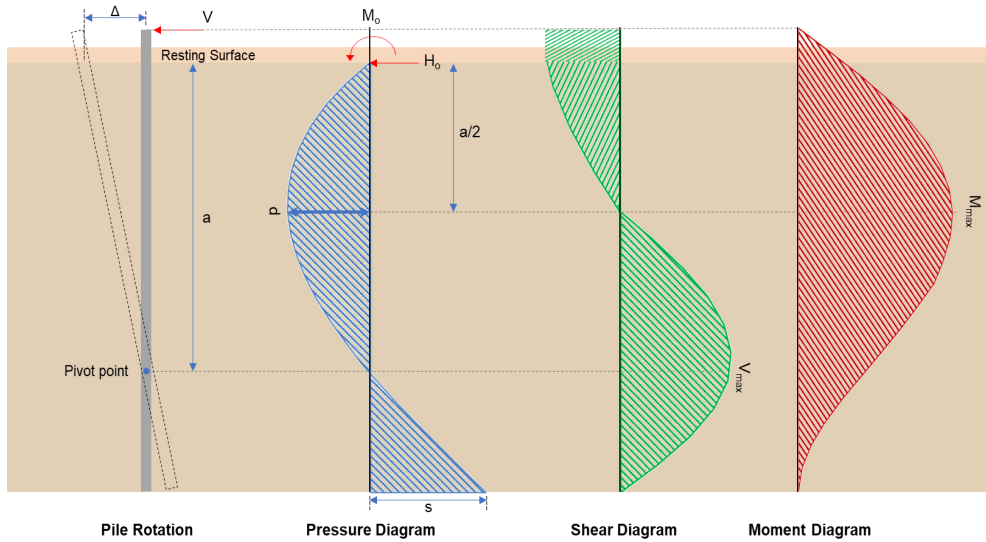
$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2291 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.992 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.64 \text{ kipft}) + ((-6.992 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.503 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.503 \text{ kipft/ft})}{(-1.1134 \text{ kip/ft})}$$

$$E = 14.823 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.503 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.1134 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (16.503 \text{ kipft/ft})) + (4 \times (-1.1134 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6861 \text{ ft}$$

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

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

$$V_{max} = ((-1.1134 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.823 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6861 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.823 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6861 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.091 \text{ kip}$$

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

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

$$M_{max} = ((-1.1134 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(14.823 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6861 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.823 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6861 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (14.823 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6861 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.186 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{12.309 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.187 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.187 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$Ratio = \frac{(12.309 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0046012$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 $b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.309 \text{ kip} \rightarrow 12309 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12309 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.13 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.13 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.13 \text{ kip}$$

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

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

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

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

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

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

$$\phi V_n = (0.65) \times ((120.13 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.16 \text{ kip}$$

Considering x-direction:

$V_{max} = 17.097 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.097 \text{ kip})}{(111.16 \text{ kip})}$$

$$\text{Ratio} = 0.1538$$

Status: **PASS**
Ratio: **0.150**

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: $M_{max} = 67.186 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(67.186 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.26918$$

Status: **PASS**
Ratio: **0.270**

REFERENCES	CALCULATIONS	RESULTS
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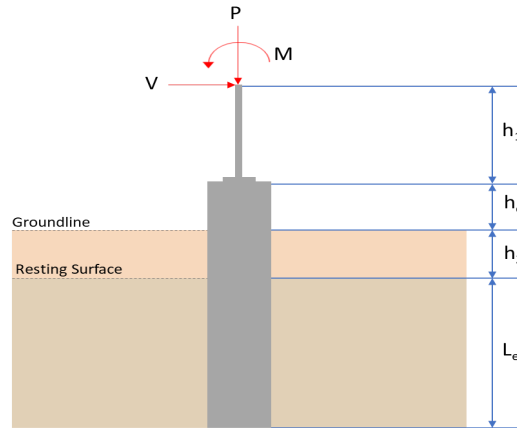
SkyCiv Foundation Design

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

$L = 8.25$ 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)	8.445	12.315
V_x (kip)	-4.151	-6.918
V_z (kip)	-0.024	-0.041
M_x (kipft)	-0.097	-0.166
M_z (kipft)	60.230	102.162

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,

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

$$H_o = \frac{(-4.151 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.66099 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(60.23 \text{ kipft}) + ((-4.151 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.5908 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.7245 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.097 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015446 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0021 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.7245 \text{ ft}), (1.0021 \text{ ft})]$$

$$L_{e,req} = 7.725 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.725 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.93636$$

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

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

$$q = \frac{(8.445 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.52781 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.52781 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26391$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.66099 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.5908 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.5908 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.5908 \text{ kipft/ft})) + (4 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.689 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.5908 \text{ kipft/ft})) + (3 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.5908 \text{ kipft/ft})) + (2 \times (-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.29862 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.5908 \text{ kipft/ft})) + ((-0.66099 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.2102 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.689 \text{ ft})}{2}$$

$$p_a = 0.42667 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29862 \text{ kip/ft}^2)}{(0.42667 \text{ kip/ft}^2)}$$

$$Ratio = 0.69987$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2102 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.97795$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = -0.0038217 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.015446 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015446 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.015446 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8963 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.015446 \text{ kipft/ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.015446 \text{ kipft/ft})) + (2 \times (-0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = -0.00070915 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.015446 \text{ kipft/ft})) + ((-0.0038217 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = -0.000056149 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8963 \text{ ft})}{2}$$

$$p_a = 0.44222 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00070915 \text{ kip/ft}^2)}{(0.44222 \text{ kip/ft}^2)}$$

$$Ratio = -0.0016036$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

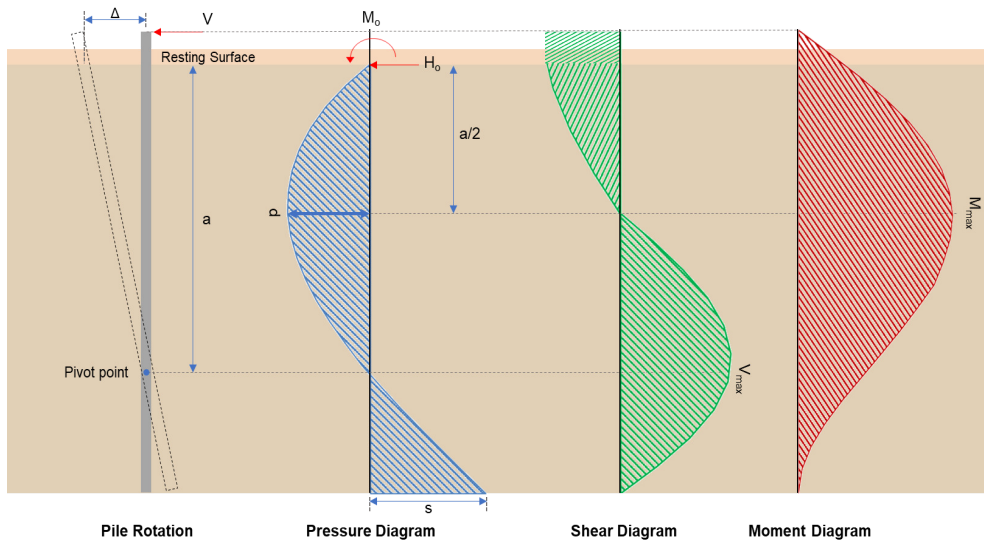
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.000056149 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = -0.000045373$$

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,

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

$$H_o = \frac{(-6.918 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1016 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(102.16 \text{ kipft}) + ((-6.918 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.268 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.268 \text{ kipft/ft})}{(-1.1016 \text{ kip/ft})}$$

$$E = 14.768 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.268 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.1016 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 16.268 \text{ kipft/ft}) + (4 \times (-1.1016 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (16.268 \text{ kipft/ft})) + (4 \times (-1.1016 \text{ kip/ft}) \times (8.25 \text{ ft}))}{(6 \times (16.268 \text{ kipft/ft})) + (4 \times (-1.1016 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6866 \text{ ft}$$

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

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

$$V_{max} = ((-1.1016 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.768 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6866 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.768 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6866 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.864 \text{ kip}$$

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

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

$$M_{max} = ((-1.1016 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(14.768 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.768 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.768 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6866 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 66.261 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.166 \text{ kipft}) + ((-0.041 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.026433 \text{ kipft/ft})}{(-0.0065287 \text{ kip/ft})}$$

$$E = 4.0488 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026433 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0065287 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.026433 \text{ kipft/ft})) + (4 \times (-0.0065287 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.896 \text{ ft}$$

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

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

$$V_{max} = ((-0.0065287 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.896 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.896 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.040082 \text{ kip}$$

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

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

$$M_{max} = ((-0.0065287 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(4.0488 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.0488 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.896 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.14613 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.315 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.187 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.187 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(12.315 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0046034$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.315 \text{ kip} \rightarrow 12315 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12315 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.13 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.13 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.13 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.13 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.16 \text{ kip}$$

Considering x-direction:

V_{max} = 16.864 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.864 \text{ kip})}{(111.16 \text{ kip})}$$

$$Ratio = 0.1517$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.040082 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.040082 \text{ kip})}{(111.16 \text{ kip})}$$

$$Ratio = 0.00036057$$

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:

$M_{max} = 66.261 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(66.261 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.26547$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 0.14613 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.14613 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00058545$$

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