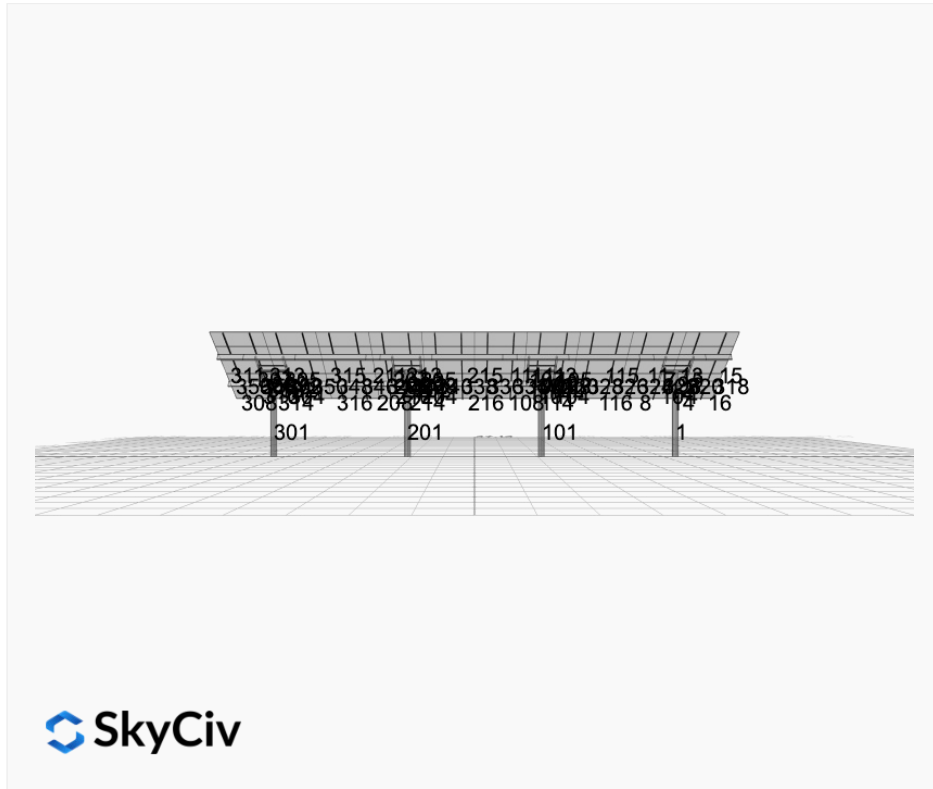


Project Details



Project Name: MTSOLAR_175CD78L72K64 - V1jb **Date:** Wed Sep 24 2025
Location: 1925 St Clair Ave NE, Cleveland, OH 44114, USA **Number of Modules:** 50
Unique ID: 4P-19.75-10TOP-XD-45-L-5Hx10W-GI33 **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.54 ft
Array Dimensions E/W	74.17 ft
Winter Tilt Angle	30
Front Edge Clearance	8 ft

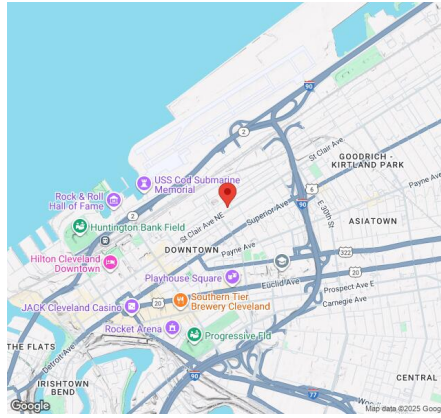
MT Solar Bill of Materials (4P-19.75-10TOP-XD-45-L-5Hx10W-GI33)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	10

Rail Bill of Materials

Part	Qty
Rails (223in)	20
Rail Attachment	80
Module Mid Clamp	80
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: 1925 St Clair Ave NE, Cleveland, OH 44114, USA

Array Specification

Duty Classification:	XD
Module Width:	44.00 in
Module Length:	88.00in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Winter Tilt Angle:	30
Front Edge Clearance:	8
Total Array Height at Tilt:	17.27 ft
Total Frame Length:	74.25 ft
Module Info/Notes:	
Array Dimensions N/S:	18.54 ft
Array Dimensions E/W:	74.17 ft
Rail Length:	222.50 in
Rail Spacing:	3.71 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	12.64 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.25 ft Pile 2: 7.75 ft Pile 3: 7.75 ft Pile 4: 7.25 ft
Foundation Volume:	17.778 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1925 St Clair Ave NE, Cleveland, OH 44114, USA
Wind Speed:	102 mph

Snow Load:

20 psf

Design Disclaimer

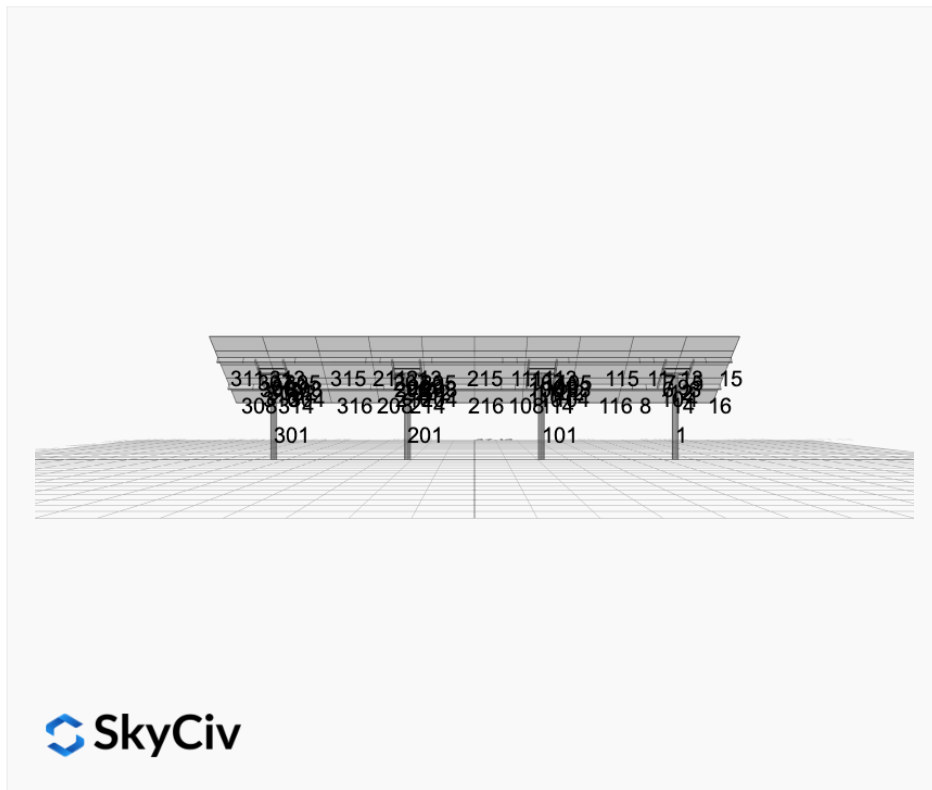
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

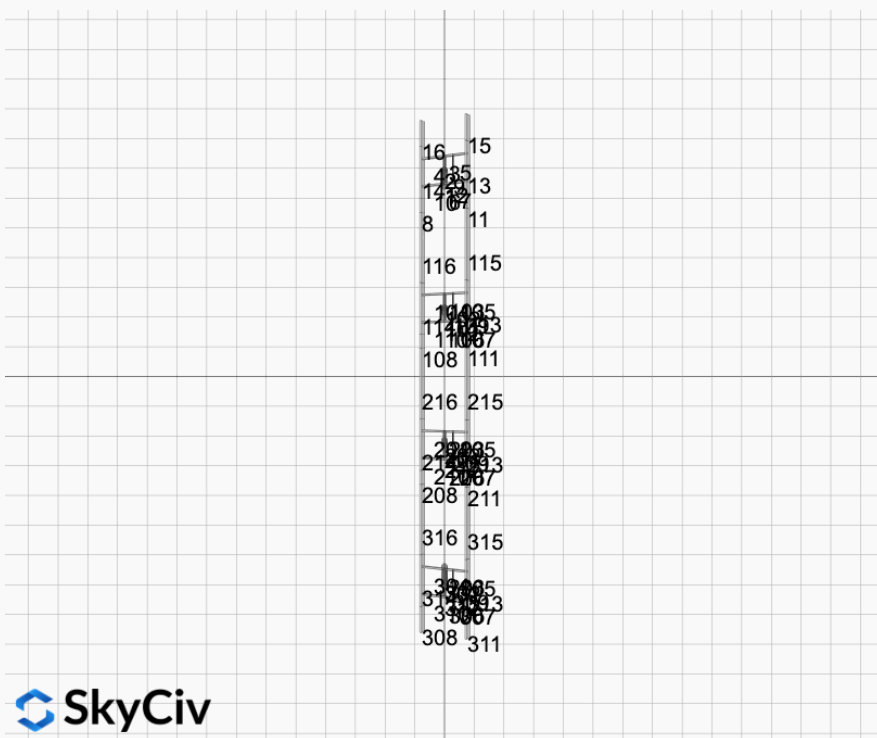
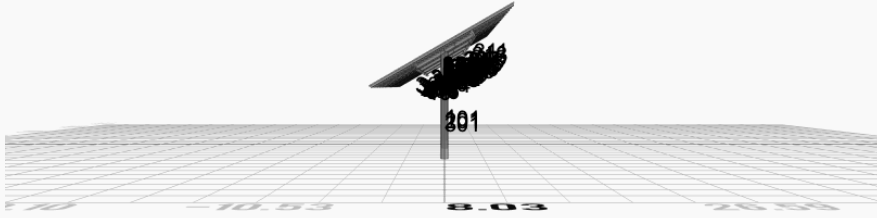
AutoDesigner Input

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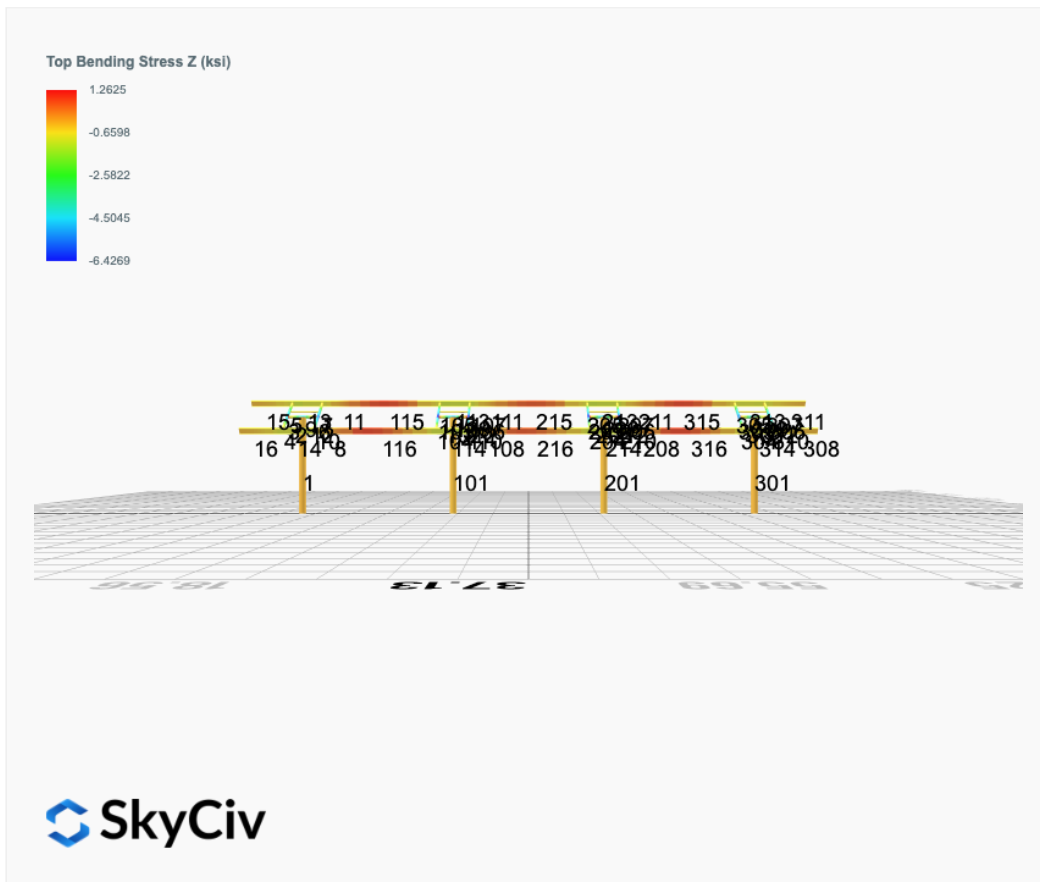
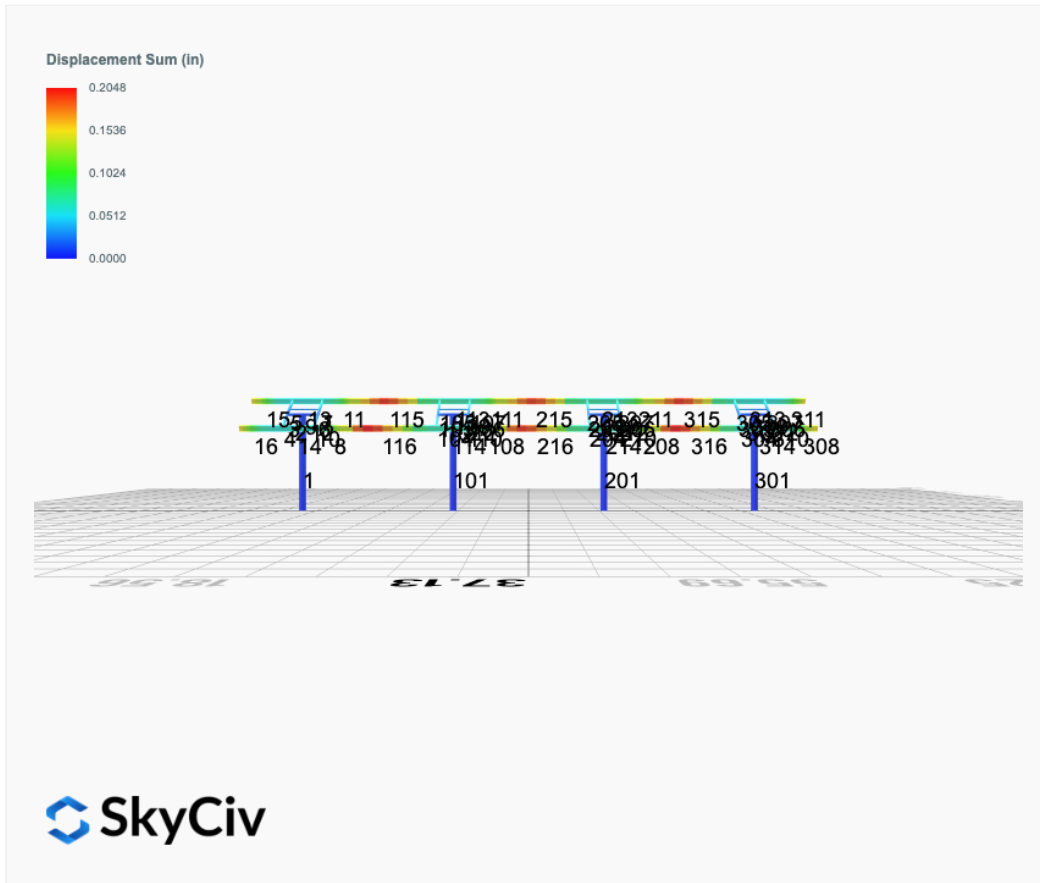
Design Notes:

- 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)

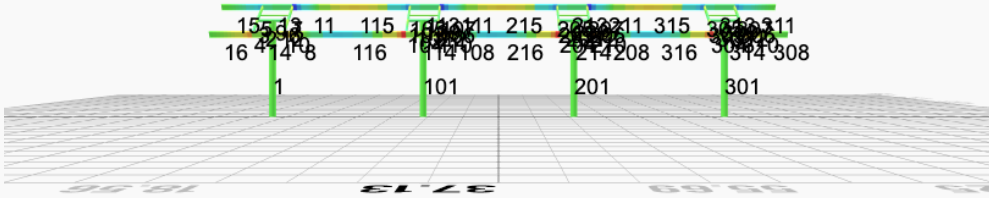
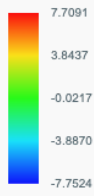




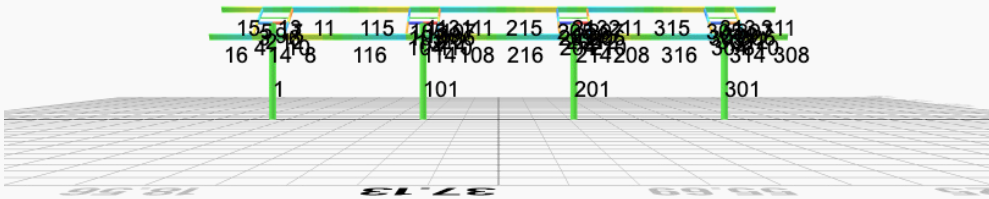
FEM Results (Envelope Worst Case for each member)



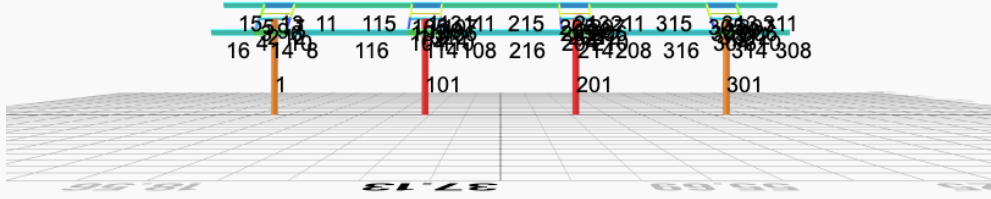
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0084	2.6129	0.0270	0.0973	0.0055	-0.0720
ULS: 2. D + L	0.0084	2.6129	0.0270	0.0973	0.0055	-0.0720
ULS: 3. D + (S or Lr or R)	0.0196	5.0458	0.0632	0.2278	0.0123	-0.2036
ULS: 3. D + (S or Lr or R)	0.0084	2.6129	0.0270	0.0973	0.0055	-0.0720
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0168	4.4376	0.0542	0.1952	0.0106	-0.1707
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0084	2.6129	0.0270	0.0973	0.0055	-0.0720
ULS: 5b. D + 0.7E	0.0084	2.6129	0.0270	0.0973	0.0055	-0.0720
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0168	4.4376	0.0542	0.1952	0.0106	-0.1707
ULS: 8. 0.6D + 0.7E	0.0050	1.5677	0.0162	0.0584	0.0033	-0.0432
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2218	8.1707	0.1336	0.4510	-0.3641	41.4961
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.2218	8.1707	0.1336	0.4510	-0.3641	41.4961
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7753	-2.1497	-0.0622	-0.1984	0.3153	-34.6764
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.3273	-1.3650	-0.0588	-0.1865	0.3098	-40.5016
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4058	8.6060	0.1341	0.4605	-0.2667	31.0054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4058	8.6060	0.1341	0.4605	-0.2667	31.0054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0920	0.8657	-0.0127	-0.0266	0.2429	-26.1240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7560	1.4541	-0.0102	-0.0177	0.2388	-30.4929
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4143	6.7813	0.1070	0.3626	-0.2717	31.1041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4143	6.7813	0.1070	0.3626	-0.2717	31.1041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0836	-0.9590	-0.0399	-0.1245	0.2378	-26.0253
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7476	-0.3706	-0.0373	-0.1155	0.2338	-30.3942
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2252	7.1256	0.1228	0.4121	-0.3663	41.5249
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.2252	7.1256	0.1228	0.4121	-0.3663	41.5249
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7720	-3.1948	-0.0730	-0.2373	0.3131	-34.6476
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.3240	-2.4102	-0.0696	-0.2254	0.3076	-40.4728

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	13.6156
Shear X	-5.3836
Shear Z	0.2292
Moment X	0.7753
Moment Y (Twist)	0.6161
Moment Z	69.6998

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	8.6060
Shear X	-3.2252
Shear Z	0.1341
Moment X	0.4605
Moment Y (Twist)	0.3663
Moment Z	41.5249

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	-0.0084	2.8972	-0.0008	-0.0032	0.0020	0.1326
ULS: 2. D + L	-0.0084	2.8972	-0.0008	-0.0032	0.0020	0.1326
ULS: 3. D + (S or Lr or R)	-0.0196	5.7085	-0.0020	-0.0075	0.0047	0.2765
ULS: 3. D + (S or Lr or R)	-0.0084	2.8972	-0.0008	-0.0032	0.0020	0.1326
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0168	5.0057	-0.0017	-0.0064	0.0040	0.2406

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0084	2.8972	-0.0008	-0.0032	0.0020	0.1326
ULS: 5b. D + 0.7E	-0.0084	2.8972	-0.0008	-0.0032	0.0020	0.1326
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0168	5.0057	-0.0017	-0.0064	0.0040	0.2406
ULS: 8. 0.6D + 0.7E	-0.0050	1.7383	-0.0005	-0.0019	0.0012	0.0796
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6933	9.3166	0.0103	0.0317	-0.0582	47.4089
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.6933	9.3166	0.0103	0.0317	-0.0582	47.4089
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1519	-2.6065	-0.0093	-0.0295	0.0490	-39.1154
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6120	-1.6801	-0.0145	-0.0465	0.0708	-45.3914
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7805	9.8203	0.0067	0.0197	-0.0411	35.6978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.7805	9.8203	0.0067	0.0197	-0.0411	35.6978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3534	0.8779	-0.0081	-0.0261	0.0393	-29.1955
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9485	1.5727	-0.0119	-0.0389	0.0556	-33.9025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7721	7.7118	0.0076	0.0230	-0.0432	35.5898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.7721	7.7118	0.0076	0.0230	-0.0432	35.5898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3618	-1.2306	-0.0072	-0.0229	0.0372	-29.3034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9569	-0.5358	-0.0111	-0.0356	0.0536	-34.0104
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6899	8.1577	0.0107	0.0330	-0.0590	47.3558
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.6899	8.1577	0.0107	0.0330	-0.0590	47.3558
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1552	-3.7654	-0.0090	-0.0282	0.0482	-39.1685
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6154	-2.8390	-0.0141	-0.0452	0.0700	-45.4445

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	15.5807
Shear X	-6.1563
Shear Z	-0.0249
Moment X	-0.0796
Moment Y (Twist)	0.1208
Moment Z	79.7294

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	9.8203
Shear X	-3.6933
Shear Z	-0.0145
Moment X	-0.0465
Moment Y (Twist)	0.0708
Moment Z	47.4089

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	-0.0084	2.8972	0.0008	0.0032	-0.0020	0.1326
ULS: 2. D + L	-0.0084	2.8972	0.0008	0.0032	-0.0020	0.1326
ULS: 3. D + (S or Lr or R)	-0.0196	5.7085	0.0020	0.0075	-0.0047	0.2765
ULS: 3. D + (S or Lr or R)	-0.0084	2.8972	0.0008	0.0032	-0.0020	0.1326
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0168	5.0057	0.0017	0.0064	-0.0040	0.2406
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0084	2.8972	0.0008	0.0032	-0.0020	0.1326
ULS: 5b. D + 0.7E	-0.0084	2.8972	0.0008	0.0032	-0.0020	0.1326
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0168	5.0057	0.0017	0.0064	-0.0040	0.2406
ULS: 8. 0.6D + 0.7E	-0.0050	1.7383	0.0005	0.0019	-0.0012	0.0796
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6933	9.3166	-0.0103	-0.0317	0.0582	47.4089
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.6933	9.3166	-0.0103	-0.0317	0.0582	47.4089
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1519	-2.6065	0.0093	0.0295	-0.0490	-39.1154
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6120	-1.6801	0.0145	0.0465	-0.0707	-45.3914

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7805	9.8203	-0.0067	-0.0197	0.0412	35.6978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.7805	9.8203	-0.0067	-0.0197	0.0412	35.6978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3534	0.8779	0.0081	0.0261	-0.0393	-29.1955
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9485	1.5727	0.0119	0.0389	-0.0556	-33.9025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7721	7.7118	-0.0075	-0.0230	0.0432	35.5898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.7721	7.7118	-0.0075	-0.0230	0.0432	35.5898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3618	-1.2306	0.0072	0.0229	-0.0372	-29.3034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9569	-0.5358	0.0111	0.0357	-0.0536	-34.0104
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6899	8.1577	-0.0107	-0.0330	0.0590	47.3558
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.6899	8.1577	-0.0107	-0.0330	0.0590	47.3558
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1552	-3.7654	0.0090	0.0282	-0.0482	-39.1685
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6154	-2.8390	0.0141	0.0452	-0.0700	-45.4445

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	15.5807
Shear X	-6.1563
Shear Z	0.0249
Moment X	0.0800
Moment Y (Twist)	0.1206
Moment Z	79.7294

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	9.8203
Shear X	-3.6933
Shear Z	0.0145
Moment X	0.0465
Moment Y (Twist)	0.0707
Moment Z	47.4089

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	0.0084	2.6129	-0.0270	-0.0973	-0.0055	-0.0720
ULS: 2. D + L	0.0084	2.6129	-0.0270	-0.0973	-0.0055	-0.0720
ULS: 3. D + (S or Lr or R)	0.0196	5.0458	-0.0632	-0.2278	-0.0122	-0.2035
ULS: 3. D + (S or Lr or R)	0.0084	2.6129	-0.0270	-0.0973	-0.0055	-0.0720
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0168	4.4376	-0.0542	-0.1952	-0.0105	-0.1707
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0084	2.6129	-0.0270	-0.0973	-0.0055	-0.0720
ULS: 5b. D + 0.7E	0.0084	2.6129	-0.0270	-0.0973	-0.0055	-0.0720
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0168	4.4376	-0.0542	-0.1952	-0.0105	-0.1707
ULS: 8. 0.6D + 0.7E	0.0050	1.5677	-0.0162	-0.0584	-0.0033	-0.0432
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2218	8.1707	-0.1336	-0.4510	0.3641	41.4961
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.2218	8.1707	-0.1336	-0.4510	0.3641	41.4961
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7753	-2.1497	0.0622	0.1984	-0.3153	-34.6764
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.3273	-1.3650	0.0588	0.1865	-0.3098	-40.5016
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4058	8.6060	-0.1341	-0.4605	0.2667	31.0054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4058	8.6060	-0.1341	-0.4605	0.2667	31.0054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0920	0.8657	0.0127	0.0266	-0.2429	-26.1240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7560	1.4541	0.0102	0.0177	-0.2388	-30.4929
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4143	6.7813	-0.1070	-0.3626	0.2717	31.1041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4143	6.7813	-0.1070	-0.3626	0.2717	31.1041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0836	-0.9590	0.0399	0.1245	-0.2378	-26.0253
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7476	-0.3706	0.0373	0.1155	-0.2337	-30.3942

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2252	7.1256	-0.1228	-0.4121	0.3663	41.5249
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.2252	7.1256	-0.1228	-0.4121	0.3663	41.5249
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7720	-3.1948	0.0730	0.2373	-0.3131	-34.6476
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.3240	-2.4102	0.0696	0.2254	-0.3076	-40.4728

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.6156
Shear X	-5.3836
Shear Z	-0.2292
Moment X	-0.7752
Moment Y (Twist)	0.6161
Moment Z	69.7005

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	8.6060
Shear X	-3.2252
Shear Z	-0.1341
Moment X	-0.4605
Moment Y (Twist)	0.3663
Moment Z	41.5249

Project Details

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

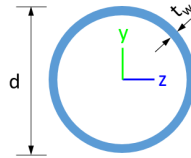


Design Input Information

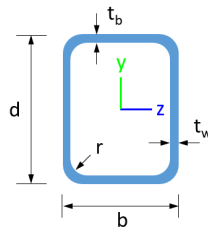
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

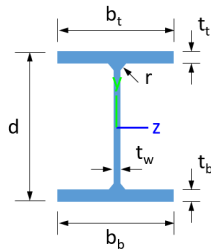
Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
11	10in Pipe Sch 40	10.75	0.36				



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



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

Section Properties

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

315	20	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.13,1.13,1.14,1.13,1.13,1.13,1.13,1.13,1.12,1.19,1.13,1.13,1.13,1.13,1.13,1.13,1.13	300	200	1
316	20	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.29,1.12,1.12,1.12,1.10,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.16,1.12,1.12,1.12,1.10	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	535.87	309.42	147.68	147.68	160.76	160.76
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	33.56	6.46	56.26	44.91
14	159.30	97.43	33.14	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	535.87	309.42	147.68	147.68	160.76	160.76
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.47	6.46	56.26	44.91
114	159.30	97.43	31.86	6.46	56.26	44.91
115	159.30	75.13	20.98	6.46	56.26	44.91
116	159.30	75.13	21.41	6.46	56.26	44.91
201	535.87	309.42	147.68	147.68	160.76	160.76
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30

212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.48	6.46	56.26	44.91
214	159.30	97.43	31.86	6.46	56.26	44.91
215	159.30	75.13	21.13	6.46	56.26	44.91
216	159.30	75.13	21.49	6.46	56.26	44.91
301	535.87	309.42	147.68	147.68	160.76	160.76
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	55.15	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	55.15	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	33.56	6.46	56.26	44.91
314	159.30	97.43	33.12	6.46	56.26	44.91
315	159.30	75.13	21.60	6.46	56.26	44.91
316	159.30	75.13	21.09	6.46	56.26	44.91

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.044	0.472	0.014	0.033	0.001	0.499	#13	0.433	Not Required	Pass
2	0.002	0.369	0.177	0.081	0.034	0.547	#13	0.036	Not Required	Pass
3	0.005	0.604	0.023	0.060	0.002	0.617	#13	0.046	Not Required	Pass
4	0.004	0.603	0.078	0.060	0.017	0.644	#13	0.082	Not Required	Pass
5	0.005	0.375	0.074	0.060	0.019	0.382	#13	0.076	Not Required	Pass
6	0.006	0.684	0.046	0.069	0.009	0.715	#13	0.046	Not Required	Pass
7	0.006	0.424	0.104	0.068	0.027	0.440	#13	0.076	Not Required	Pass
8	0.001	0.076	0.112	0.042	0.012	0.116	#24	0.102	Not Required	Pass
9	0.008	0.067	0.047	0.002	0.001	0.116	#13	0.206	Not Required	Pass
10	0.006	0.671	0.102	0.067	0.022	0.694	#13	0.082	Not Required	Pass
11	0.002	0.073	0.113	0.043	0.012	0.118	#24	0.102	Not Required	Pass
12	0.002	0.447	0.200	0.092	0.037	0.647	#13	0.054	Not Required	Pass
13	0.004	0.204	0.283	0.055	0.016	0.389	#21	0.306	Not Required	Pass
14	0.005	0.202	0.281	0.054	0.016	0.378	#21	0.204	Not Required	Pass
15	0.000	0.059	0.096	0.026	0.007	0.142	#21	Not Required	Not Required	Pass
16	0.000	0.059	0.096	0.026	0.007	0.142	#21	Not Required	Not Required	Pass
101	0.050	0.540	0.002	0.038	0.000	0.565	#13	0.433	Not Required	Pass
102	0.002	0.466	0.215	0.099	0.040	0.682	#13	0.036	Not Required	Pass
103	0.006	0.732	0.037	0.073	0.005	0.756	#13	0.046	Not Required	Pass
104	0.006	0.738	0.102	0.074	0.022	0.782	#13	0.082	Not Required	Pass
105	0.006	0.454	0.105	0.073	0.027	0.470	#13	0.076	Not Required	Pass
106	0.006	0.742	0.038	0.074	0.006	0.764	#13	0.046	Not Required	Pass
107	0.006	0.460	0.101	0.074	0.026	0.476	#13	0.076	Not Required	Pass
108	0.002	0.053	0.105	0.043	0.012	0.133	#21	0.102	Not Required	Pass
109	0.009	0.067	0.041	0.001	0.000	0.109	#13	0.206	Not Required	Pass
110	0.006	0.741	0.098	0.074	0.021	0.779	#13	0.082	Not Required	Pass

111	0.002	0.061	0.107	0.043	0.012	0.131	#21	0.102	Not Required	Pass
112	0.002	0.473	0.219	0.100	0.041	0.692	#13	0.036	Not Required	Pass
113	0.004	0.214	0.285	0.056	0.016	0.433	#21	0.306	Not Required	Pass
114	0.006	0.233	0.284	0.057	0.016	0.440	#21	0.306	Not Required	Pass
115	0.003	0.302	0.151	0.044	0.012	0.388	#13	0.507	Not Required	Pass
116	0.002	0.295	0.152	0.045	0.012	0.382	#13	0.507	Not Required	Pass
201	0.050	0.540	0.002	0.038	0.000	0.565	#13	0.433	Not Required	Pass
202	0.002	0.473	0.219	0.100	0.041	0.692	#13	0.036	Not Required	Pass
203	0.006	0.742	0.038	0.074	0.006	0.764	#13	0.046	Not Required	Pass
204	0.006	0.741	0.098	0.074	0.021	0.779	#13	0.082	Not Required	Pass
205	0.006	0.460	0.101	0.074	0.026	0.476	#13	0.076	Not Required	Pass
206	0.006	0.732	0.037	0.073	0.005	0.756	#13	0.046	Not Required	Pass
207	0.006	0.454	0.105	0.073	0.027	0.470	#13	0.076	Not Required	Pass
208	0.001	0.062	0.114	0.045	0.012	0.137	#21	0.102	Not Required	Pass
209	0.009	0.067	0.041	0.001	0.000	0.109	#13	0.206	Not Required	Pass
210	0.006	0.738	0.102	0.074	0.022	0.782	#13	0.082	Not Required	Pass
211	0.002	0.069	0.115	0.044	0.012	0.134	#21	0.102	Not Required	Pass
212	0.002	0.466	0.215	0.099	0.040	0.682	#13	0.036	Not Required	Pass
213	0.004	0.214	0.285	0.056	0.016	0.433	#21	0.306	Not Required	Pass
214	0.006	0.233	0.284	0.057	0.016	0.440	#21	0.306	Not Required	Pass
215	0.003	0.278	0.151	0.043	0.012	0.363	#21	0.507	Not Required	Pass
216	0.002	0.259	0.151	0.043	0.012	0.352	#21	0.507	Not Required	Pass
301	0.044	0.472	0.014	0.033	0.001	0.499	#13	0.433	Not Required	Pass
302	0.002	0.447	0.200	0.092	0.037	0.647	#13	0.054	Not Required	Pass
303	0.006	0.684	0.046	0.069	0.009	0.715	#13	0.046	Not Required	Pass
304	0.006	0.671	0.102	0.067	0.022	0.694	#13	0.082	Not Required	Pass
305	0.006	0.424	0.104	0.068	0.027	0.440	#13	0.076	Not Required	Pass
306	0.005	0.604	0.023	0.060	0.002	0.617	#13	0.046	Not Required	Pass
307	0.005	0.375	0.074	0.060	0.019	0.382	#13	0.076	Not Required	Pass
308	0.000	0.059	0.096	0.026	0.007	0.142	#21	Not Required	Not Required	Pass
309	0.008	0.067	0.047	0.002	0.001	0.116	#13	0.206	Not Required	Pass
310	0.004	0.603	0.078	0.060	0.017	0.644	#13	0.082	Not Required	Pass
311	0.000	0.059	0.096	0.026	0.007	0.142	#21	Not Required	Not Required	Pass
312	0.002	0.369	0.177	0.081	0.034	0.547	#13	0.036	Not Required	Pass
313	0.004	0.204	0.283	0.055	0.016	0.389	#21	0.204	Not Required	Pass
314	0.005	0.202	0.281	0.054	0.016	0.378	#21	0.306	Not Required	Pass
315	0.003	0.304	0.151	0.043	0.012	0.388	#13	0.507	Not Required	Pass
316	0.002	0.302	0.151	0.042	0.012	0.387	#13	0.507	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS
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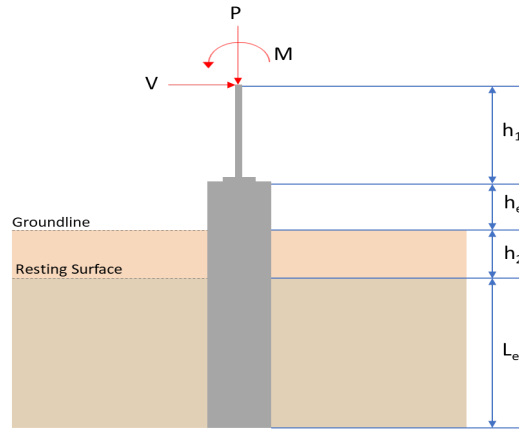
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.606	13.616
V_x (kip)	-3.225	-5.384
V_z (kip)	0.134	0.229
M_x (kipft)	0.460	0.775
M_z (kipft)	41.525	69.700

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,

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

$$H_o = \frac{(-3.225 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.51354 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.525 \text{ kipft}) + ((-3.225 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.6123 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8295 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.134 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.021338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.46 \text{ kipft}) + ((0.134 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.073248 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0384 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.8295 \text{ ft}), (2.0384 \text{ ft})]$$

$$L_{e,req} = 6.83 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.83 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.94207$$

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

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

$$q = \frac{(8.606 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53787 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53787 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26894$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.51354 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.6123 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6123 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.6123 \text{ kipft/ft})) + (4 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9982 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.6123 \text{ kipft/ft})) + (3 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.6123 \text{ kipft/ft})) + (2 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.26886 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.6123 \text{ kipft/ft})) + ((-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0846 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9982 \text{ ft})}{2}$$

$$p_a = 0.37487 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26886 \text{ kip/ft}^2)}{(0.37487 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.71721$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0846 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99732$$

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

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

Considering z-direction:

$H_o = 0.021338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.073248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.073248 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.021338 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.073248 \text{ kipft/ft})) + (4 \times (0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1866 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.073248 \text{ kipft/ft})) + (3 \times (0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.073248 \text{ kipft/ft})) + (2 \times (0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.015456 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.073248 \text{ kipft/ft})) + ((0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.034381 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1866 \text{ ft})}{2}$$

$$p_a = 0.38899 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.015456 \text{ kip/ft}^2)}{(0.38899 \text{ kip/ft}^2)}$$

$$Ratio = 0.039734$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

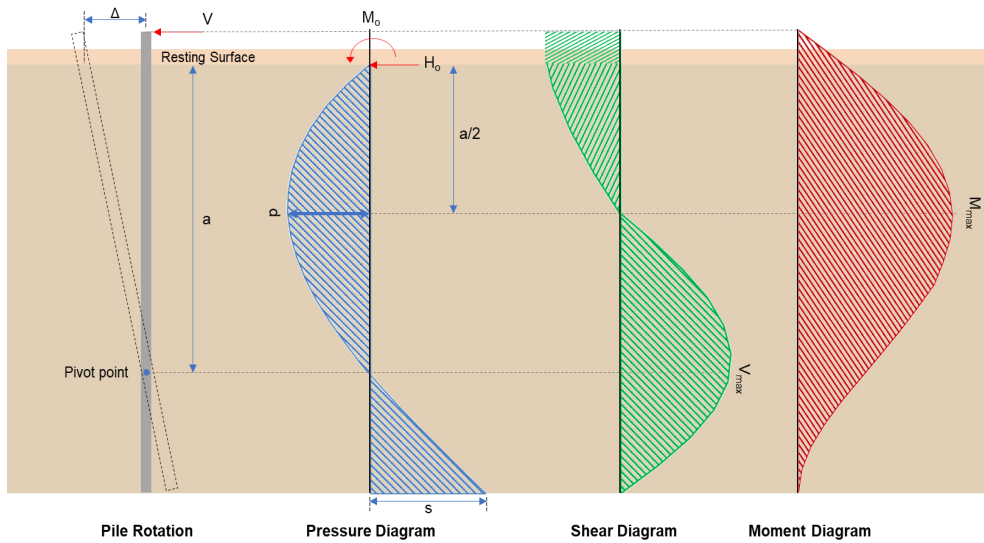
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.034381 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.031615$$

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,

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

$$H_o = \frac{(-5.384 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.85732 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.7 \text{ kipft}) + ((-5.384 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.099 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.099 \text{ kipft/ft})}{(-0.85732 \text{ kip/ft})}$$

$$E = 12.946 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.099 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.85732 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times 11.099) + (4 \times (-0.85732) \times 7.25)}$$

$$a = \frac{(-0.85732 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.099 \text{ kip/ft})) + (4 \times (-0.85732 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9976 \text{ ft}$$

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

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

$$V_{max} = ((-0.85732 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9976 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9976 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.098 \text{ kip}$$

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

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

$$M_{max} = ((-0.85732 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(12.946 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 45.221 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.229 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.036465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.775 \text{ kipft}) + ((0.229 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12341 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.12341 \text{ kipft/ft})}{(0.036465 \text{ kip/ft})}$$

$$E = 3.3843 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12341 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.036465 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.12341 \text{ kipft/ft})) + (4 \times (0.036465 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1887 \text{ ft}$$

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

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

$$V_{max} = ((0.036465 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1887 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1887 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21776 \text{ kip}$$

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

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

$$M_{max} = ((0.036465 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.3843 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.69516 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.616 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.144 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.144 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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;">$Ratio = \frac{(13.616 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0050897$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.616 \text{ kip} \rightarrow 13616 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13616 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.3 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.3 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.3 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.3 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

V_{max} = 13.098 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.098 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.1177$$

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

Considering z-direction:

$V_{max} = 0.21776 \text{ kip}$ - Maximum shear force in the z-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(0.21776 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.001957$$

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:

$M_{max} = 45.221 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(45.221 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.18117$$

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

Considering z-direction:

$M_{max} = 0.69516 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.69516 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0027851$$

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

REFERENCES	CALCULATIONS	RESULTS
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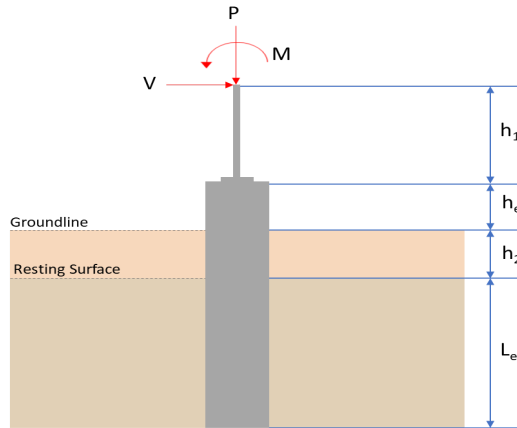
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
 $b = 48$ in - Pile width
 $D = 48$ in - Pile depth
 $L = 7.25$ ft - Total pile length
 $h_1 = 0$ ft - Lateral load height from the top of the pile,
 $h_2 = 0$ ft - Depth to resisting surface
 $h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.606	13.616
V_x (kip)	-3.225	-5.384
V_z (kip)	-0.134	-0.229
M_x (kipft)	-0.460	-0.775
M_z (kipft)	41.525	69.700

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,

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

$$H_o = \frac{(-3.225 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.51354 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.525 \text{ kipft}) + ((-3.225 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.6123 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8295 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.134 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.021338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.46 \text{ kipft}) + ((-0.134 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.073248 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5677 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.8295 \text{ ft}), (1.5677 \text{ ft})]$$

$$L_{e,req} = 6.83 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(6.83 \text{ ft})}{(7.25 \text{ ft})}$$

$$Ratio = 0.94207$$

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

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

$$q = \frac{(8.606 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53787 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53787 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26894$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.51354 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.6123 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6123 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.6123 \text{ kipft/ft})) + (4 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9982 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.6123 \text{ kipft/ft})) + (3 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.6123 \text{ kipft/ft})) + (2 \times (-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.26886 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.6123 \text{ kipft/ft})) + ((-0.51354 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0846 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9982 \text{ ft})}{2}$$

$$p_a = 0.37487 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26886 \text{ kip/ft}^2)}{(0.37487 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.71721$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0846 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99732$$

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

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

Considering z-direction:

$H_o = -0.021338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.073248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.073248 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.021338 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.073248 \text{ kipft/ft})) + (4 \times (-0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1866 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.073248 \text{ kipft/ft})) + (3 \times (-0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.073248 \text{ kipft/ft})) + (2 \times (-0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.0046594 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.073248 \text{ kipft/ft})) + ((-0.021338 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.0009361 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1866 \text{ ft})}{2}$$

$$p_a = 0.38899 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0046594 \text{ kip/ft}^2)}{(0.38899 \text{ kip/ft}^2)}$$

$$Ratio = -0.011978$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

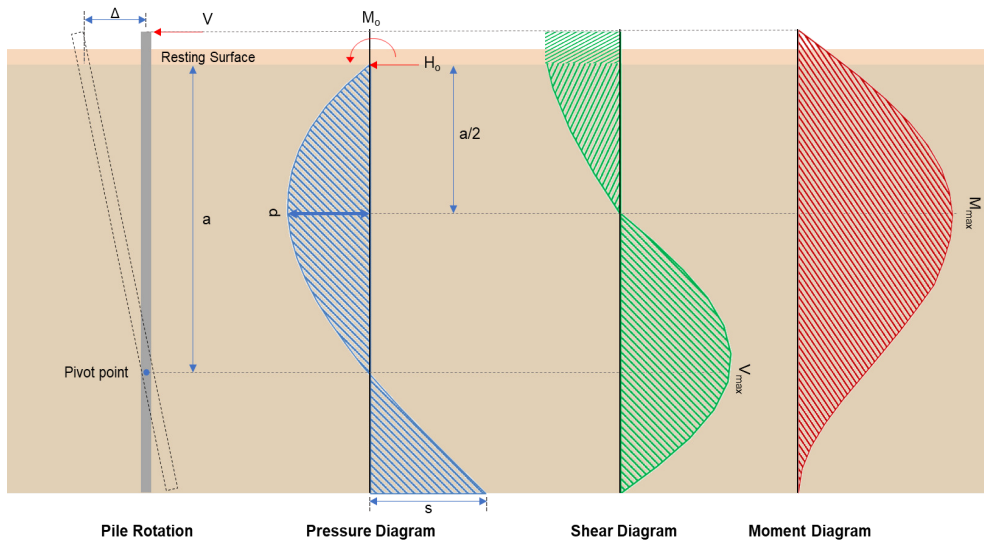
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0009361 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00086078$$

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,

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

$$H_o = \frac{(-5.384 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.85732 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(69.7 \text{ kipft}) + ((-5.384 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.099 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.099 \text{ kipft/ft})}{(-0.85732 \text{ kip/ft})}$$

$$E = 12.946 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.099 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.85732 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times 11.099) + (4 \times (-0.85732) \times 7.25)}$$

$$a = \frac{(-0.85732 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.099 \text{ kipft/ft})) + (4 \times (-0.85732 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9976 \text{ ft}$$

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

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

$$V_{max} = ((-0.85732 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9976 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9976 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.098 \text{ kip}$$

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

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

$$M_{max} = ((-0.85732 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(12.946 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9976 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 45.221 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.229 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.036465 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.775 \text{ kipft}) + ((-0.229 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12341 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.12341 \text{ kipft/ft})}{(-0.036465 \text{ kip/ft})}$$

$$E = 3.3843 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12341 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.036465 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.12341 \text{ kipft/ft})) + (4 \times (-0.036465 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1887 \text{ ft}$$

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

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

$$V_{max} = ((-0.036465 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1887 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1887 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.21776 \text{ kip}$$

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

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

$$M_{max} = ((-0.036465 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.3843 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3843 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1887 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.69516 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.616 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.144 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.144 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(13.616 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0050897$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.616 \text{ kip} \rightarrow 13616 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13616 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.3 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.3 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.3 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.3 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

V_{max} = 13.098 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.098 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.1177$$

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

Considering z-direction:

$V_{max} = 0.21776 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.21776 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.001957$$

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:

$M_{max} = 45.221 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(45.221 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.18117$$

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

Considering z-direction:

$M_{max} = 0.69516 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.69516 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0027851$$

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

REFERENCES	CALCULATIONS	RESULTS
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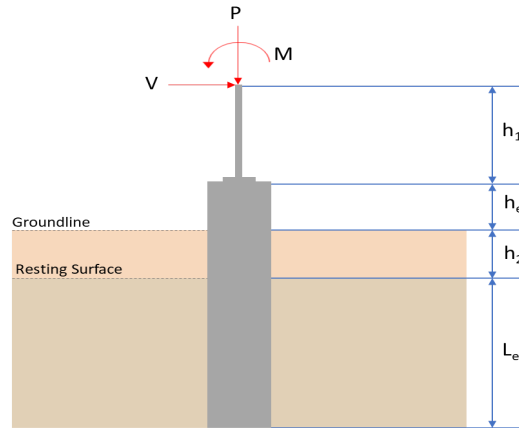
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.820	15.581
V_x (kip)	-3.693	-6.156
V_z (kip)	-0.014	-0.025
M_x (kipft)	-0.046	-0.080
M_z (kipft)	47.409	79.729

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,

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

$$H_o = \frac{(-3.693 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58806 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(47.409 \text{ kipft}) + ((-3.693 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.5492 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.0759 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.046 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0073248 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.7834 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.0759 \text{ ft}), (0.7834 \text{ ft})]$$

$$L_{e,req} = 7.076 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.076 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.91303$$

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

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

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

$$q = \frac{(9.82 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.61375 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.30687$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58806 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.5492 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.5492 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (7.5492 \text{ kipft/ft})) + (4 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.352 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.5492 \text{ kipft/ft})) + (3 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (7.5492 \text{ kipft/ft})) + (2 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.25196 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.5492 \text{ kipft/ft})) + ((-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.053 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.352 \text{ ft})}{2}$$

$$p_a = 0.4014 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.25196 \text{ kip/ft}^2)}{(0.4014 \text{ kip/ft}^2)}$$

$$Ratio = 0.6277$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.053 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.90581$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.0022293 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0073248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0073248 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.0073248 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.5614 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0073248 \text{ kipft/ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.0073248 \text{ kipft/ft})) + (2 \times (-0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = -0.00050395 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0073248 \text{ kipft/ft})) + ((-0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = -0.00026247 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5614 \text{ ft})}{2}$$

$$p_a = 0.41711 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00050395 \text{ kip/ft}^2)}{(0.41711 \text{ kip/ft}^2)}$$

$$Ratio = -0.0012082$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

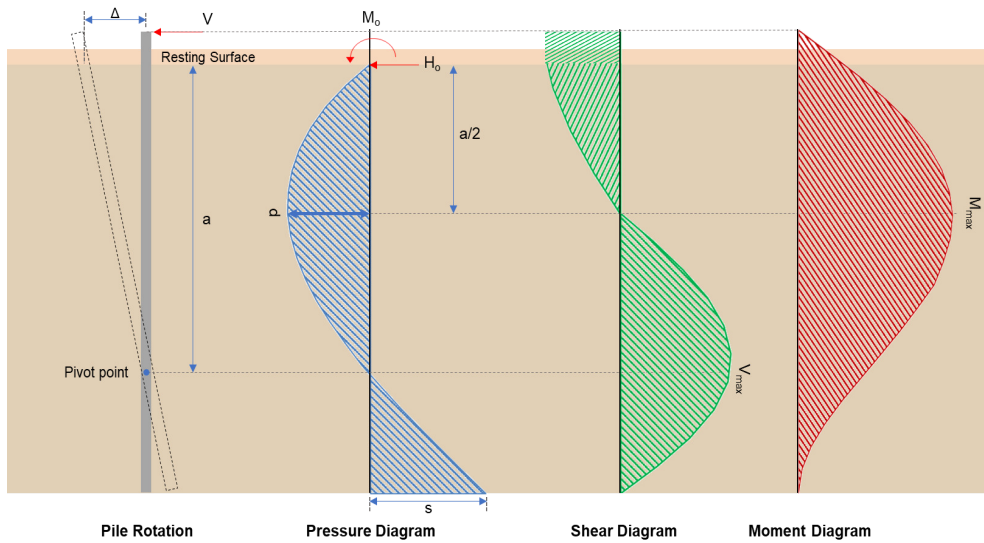
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00026247 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = -0.00022578$$

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,

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

$$H_o = \frac{(-6.156 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.98025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(79.729 \text{ kipft}) + ((-6.156 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.696 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.696 \text{ kipft/ft})}{(-0.98025 \text{ kip/ft})}$$

$$E = 12.951 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.696 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (12.696 \text{ kipft/ft})) + (4 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = \frac{(-0.98025 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.696 \text{ kip/ft})) + (4 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3508 \text{ ft}$$

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

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

$$V_{max} = ((-0.98025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3508 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3508 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.181 \text{ kip}$$

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

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

$$M_{max} = ((-0.98025 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(12.951 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 52.193 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.025 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0039809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.08 \text{ kipft}) + ((-0.025 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.012739 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.012739 \text{ kipft/ft})}{(-0.0039809 \text{ kip/ft})}$$

$$E = 3.2 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012739 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.0039809 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.012739 \text{ kipft/ft})) + (4 \times (-0.0039809 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.5655 \text{ ft}$$

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

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

$$V_{max} = ((-0.0039809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.5655 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.5655 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.022275 \text{ kip}$$

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

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

$$M_{max} = ((-0.0039809 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(3.2 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.075336 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.581 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.078 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.078 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(15.581 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0058243$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.581 \text{ kip} \rightarrow 15581 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15581 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.56 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.56 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.56 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.56 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.45 \text{ kip}$$

Considering x-direction:

V_{max} = 14.181 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.181 \text{ kip})}{(111.45 \text{ kip})}$$

$$Ratio = 0.12724$$

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

Considering z-direction:

$V_{max} = 0.022275 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.022275 \text{ kip})}{(111.45 \text{ kip})}$$

$$Ratio = 0.00019987$$

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:

$M_{max} = 52.193 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(52.193 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2091$$

Status: **PASS**
Ratio: **0.210**

Considering z-direction:

$M_{max} = 0.075336 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.075336 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00030183$$

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

REFERENCES	CALCULATIONS	RESULTS
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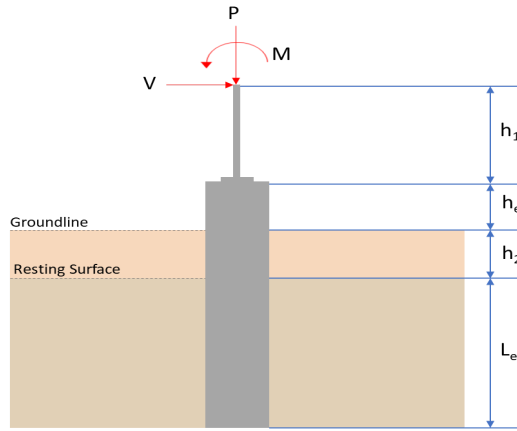
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.820	15.581
V_x (kip)	-3.693	-6.156
V_z (kip)	0.014	0.025
M_x (kipft)	0.046	0.080
M_z (kipft)	47.409	79.729

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,

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

$$H_o = \frac{(-3.693 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58806 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(47.409 \text{ kipft}) + ((-3.693 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.5492 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.0759 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.046 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0073248 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.88987 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.0759 \text{ ft}), (0.88987 \text{ ft})]$$

$$L_{e,req} = 7.076 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.076 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.91303$$

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

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

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

$$q = \frac{(9.82 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.61375 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61375 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.30687$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58806 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.5492 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.5492 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (7.5492 \text{ kipft/ft})) + (4 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.352 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.5492 \text{ kipft/ft})) + (3 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (7.5492 \text{ kipft/ft})) + (2 \times (-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.25196 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.5492 \text{ kipft/ft})) + ((-0.58806 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.053 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.352 \text{ ft})}{2}$$

$$p_a = 0.4014 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25196 \text{ kip/ft}^2)}{(0.4014 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.6277$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.053 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90581$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.0022293 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0073248 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0073248 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.0073248 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.5614 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0073248 \text{ kipft/ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.0073248 \text{ kipft/ft})) + (2 \times (0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.001454 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0073248 \text{ kipft/ft})) + ((0.0022293 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 0.0031894 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5614 \text{ ft})}{2}$$

$$p_a = 0.41711 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.001454 \text{ kip/ft}^2)}{(0.41711 \text{ kip/ft}^2)}$$

$$Ratio = 0.0034859$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

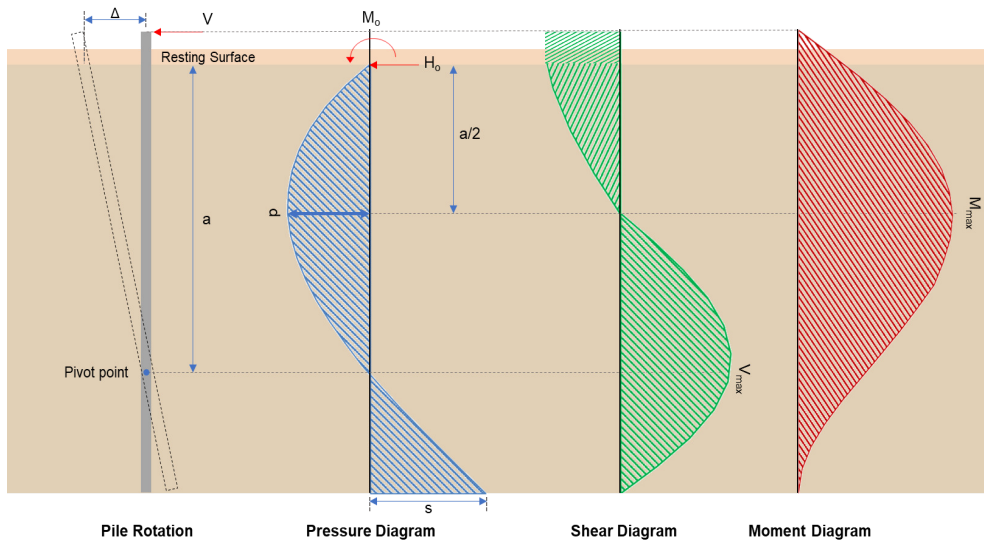
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0031894 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0027435$$

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,

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

$$H_o = \frac{(-6.156 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.98025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(79.729 \text{ kipft}) + ((-6.156 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.696 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.696 \text{ kipft/ft})}{(-0.98025 \text{ kip/ft})}$$

$$E = 12.951 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.696 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (12.696 \text{ kipft/ft})) + (4 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = \frac{(-0.98025 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.696 \text{ kip/ft})) + (4 \times (-0.98025 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3508 \text{ ft}$$

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

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

$$V_{max} = ((-0.98025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3508 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3508 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.181 \text{ kip}$$

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

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

$$M_{max} = ((-0.98025 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(12.951 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.951 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3508 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 52.193 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.025 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0039809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.08 \text{ kipft}) + ((0.025 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.012739 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.012739 \text{ kipft/ft})}{(0.0039809 \text{ kip/ft})}$$

$$E = 3.2 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012739 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.0039809 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.012739 \text{ kipft/ft})) + (4 \times (0.0039809 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.5655 \text{ ft}$$

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

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

$$V_{max} = ((0.0039809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.5655 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.5655 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.022275 \text{ kip}$$

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

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

$$M_{max} = ((0.0039809 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(3.2 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.5655 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.075336 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.581 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.078 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.078 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(15.581 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0058243$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.581 \text{ kip} \rightarrow 15581 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15581 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.56 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.56 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.56 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.56 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.45 \text{ kip}$$

Considering x-direction:

V_{max} = 14.181 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.181 \text{ kip})}{(111.45 \text{ kip})}$$

$$Ratio = 0.12724$$

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

Considering z-direction:

$V_{max} = 0.022275 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.022275 \text{ kip})}{(111.45 \text{ kip})}$$

$$Ratio = 0.00019987$$

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:

$M_{max} = 52.193 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(52.193 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2091$$

Status: **PASS**
Ratio: **0.210**

Considering z-direction:

$M_{max} = 0.075336 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.075336 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00030183$$

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