

Project Details



Project Name: AZ-BW-Safford-4x12

Date: Mon Nov 04 2024

Location: 1391 W Thatcher Blvd, Safford, AZ 85546, USA

Number of Modules: 48

Unique ID: 5P-19.75-6TOP-SD-24-L-4Hx12W-2343

Number of Poles: 5

Dealer: _____

Date Sold: _____



Array Dimensions N/S	15.05 ft
Array Dimensions E/W	90.69 ft
Winter Tilt Angle	5
Front Edge Clearance	10 ft

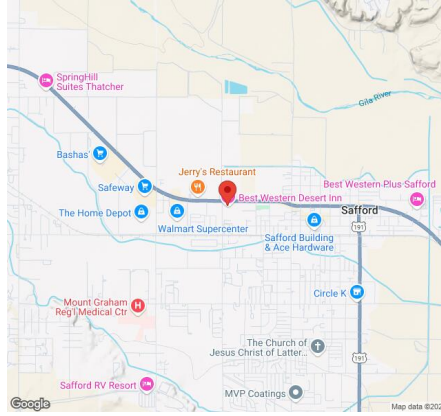
MT Solar Bill of Materials (5P-19.75-6TOP-SD-24-L-4Hx12W-2343)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-SD	H-Frame Assembly-SD	5
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	8
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	12

Rail Bill of Materials

Part	Qty
Rails (179in)	24
Rail Attachment	48
Module Mid Clamp	72
Module End Clamp	48
Ground Lug	12

Site Details:



Site Address: 1391 W Thatcher Blvd, Safford, AZ 85546, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	4
Number of Columns:	12
Total Number of Modules:	48
Winter Tilt Angle:	5
Front Edge Clearance:	10
Total Array Height at Tilt:	11.31 ft
Total Frame Length:	90.50 ft
Frame Weight:	4421 lbs
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	90.69 ft
Rail Length:	180.60 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.66 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.25 ft Pile 2: 4.50 ft Pile 3: 4.50 ft Pile 4: 4.50 ft Pile 5: 4.25 ft
Foundation Volume:	13.037 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1391 W Thatcher Blvd, Safford, AZ 85546, USA
Wind Speed:	96 mph

Snow Load:

0 psf

Design Disclaimer

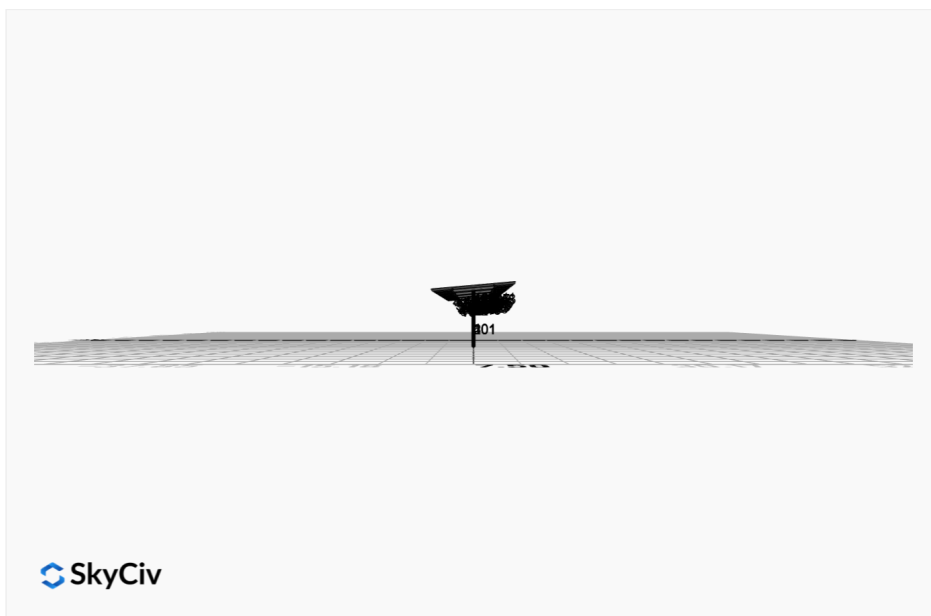
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

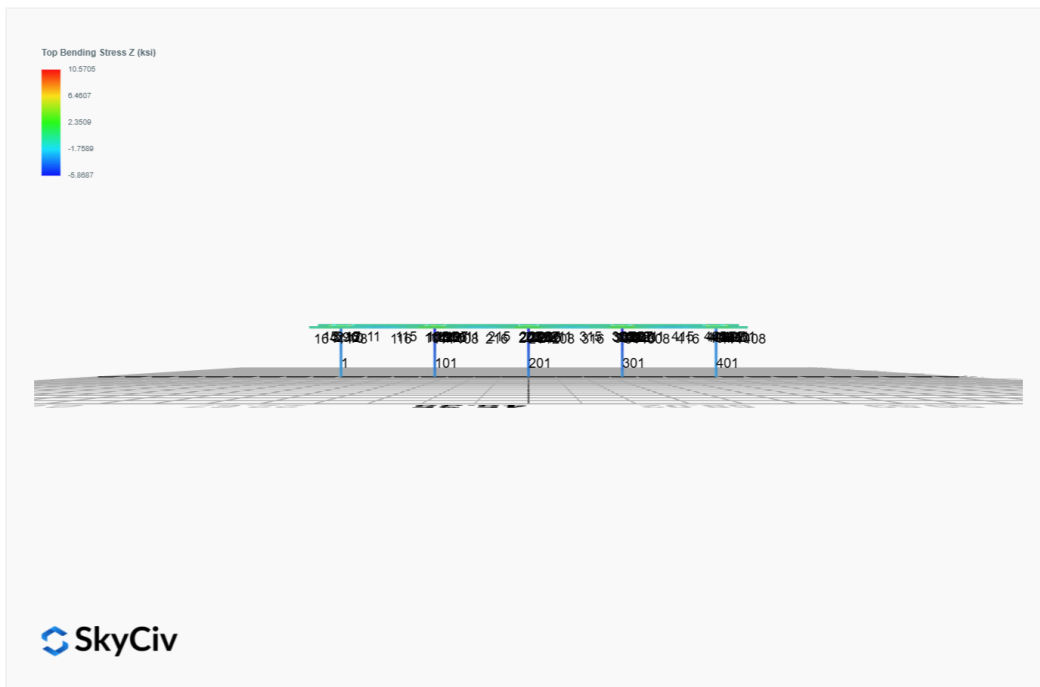
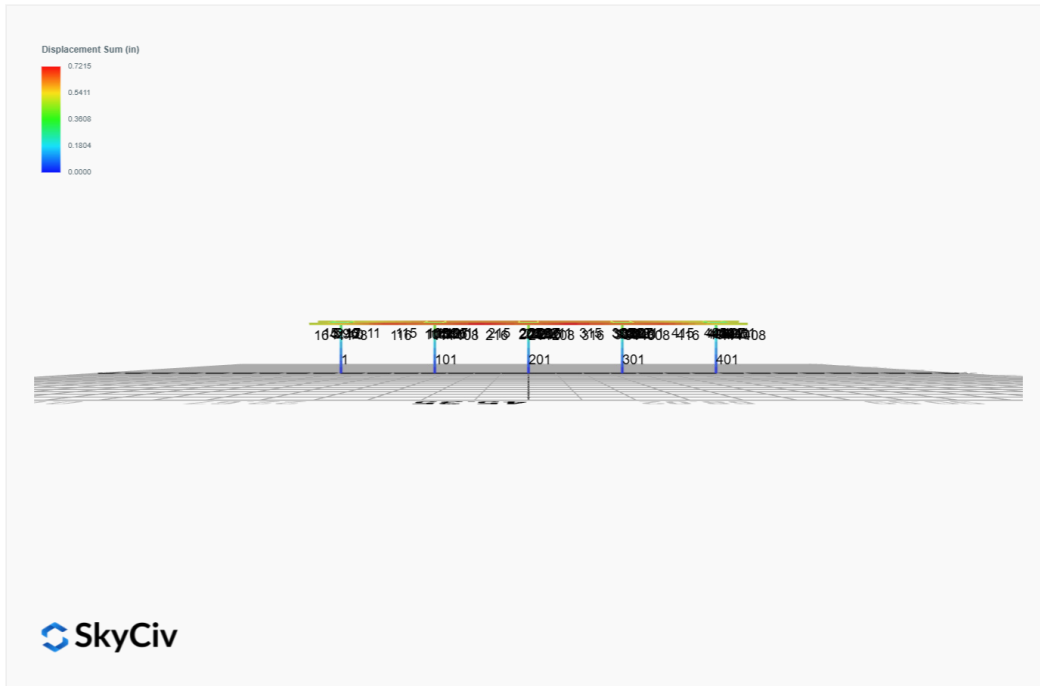
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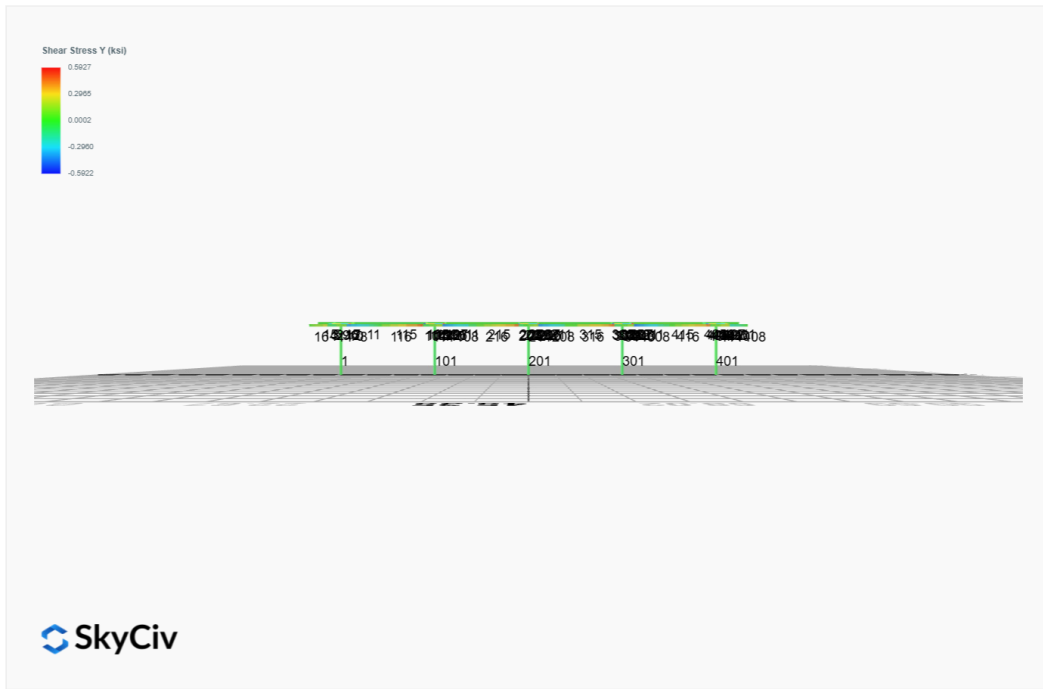
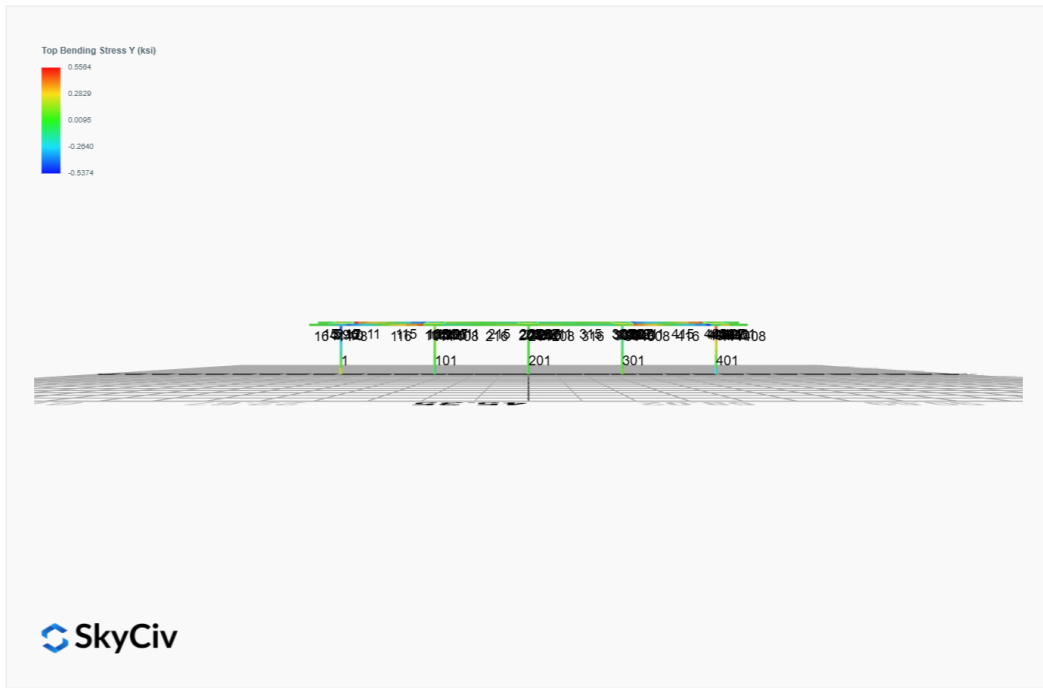
Design Notes:

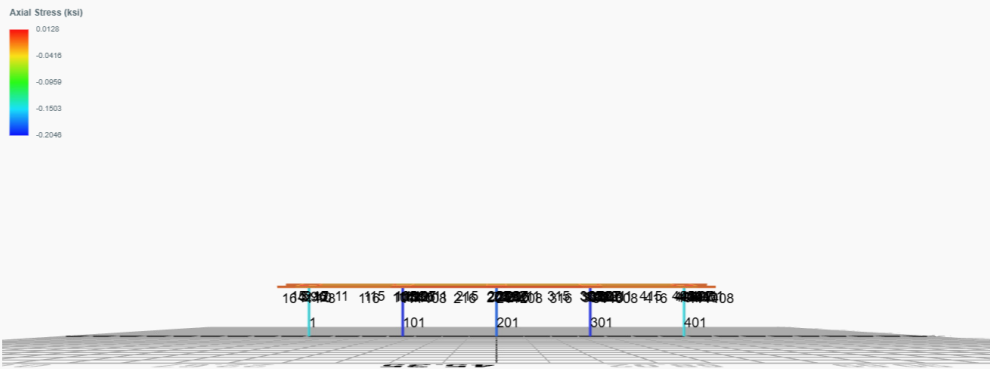
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigned are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel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



FEM Results (Envelope Worst Case for each member)







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	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 2. D + L	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 3. D + (S or Lr or R)	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 3. D + (S or Lr or R)	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 5b. D + 0.7E	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0042	1.6431	0.0809	0.2635	-0.0129	-0.0190
ULS: 8. 0.6D + 0.7E	0.0025	0.9858	0.0485	0.1581	-0.0078	-0.0114
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1652	3.4835	0.1979	0.6431	-0.0515	2.2004
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1652	3.4835	0.1979	0.6431	-0.0515	2.2004
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0401	1.1472	0.0514	0.1673	-0.0095	1.5212
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1306	0.4733	0.0033	0.0134	0.0248	-5.6176
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1229	3.0234	0.1686	0.5482	-0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1229	3.0234	0.1686	0.5482	-0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0311	1.2711	0.0587	0.1914	-0.0103	1.1361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0990	0.7657	0.0227	0.0759	0.0153	-4.2179
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1229	3.0234	0.1686	0.5482	-0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1229	3.0234	0.1686	0.5482	-0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0311	1.2711	0.0587	0.1914	-0.0103	1.1361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0990	0.7657	0.0227	0.0759	0.0153	-4.2179
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1669	2.8263	0.1655	0.5377	-0.0463	2.2080
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1669	2.8263	0.1655	0.5377	-0.0463	2.2080
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0384	0.4899	0.0190	0.0619	-0.0043	1.5288
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1290	-0.1840	-0.0290	-0.0920	0.0299	-5.6100

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	5.0389
Shear X	-0.2824
Shear Z	0.2927
Moment X	0.9523
Moment Y (Twist)	0.0802
Moment Z	9.5405

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	3.4835
Shear X	-0.1669
Shear Z	0.1979
Moment X	0.6431
Moment Y (Twist)	0.0515
Moment Z	5.6176

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	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 2. D + L	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 3. D + (S or Lr or R)	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 3. D + (S or Lr or R)	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 5b. D + 0.7E	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0046	2.1300	-0.0130	-0.0436	0.0030	0.0629
ULS: 8. 0.6D + 0.7E	-0.0028	1.2780	-0.0078	-0.0262	0.0018	0.0378
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2183	4.6650	-0.0310	-0.1051	0.0024	2.8335
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2183	4.6650	-0.0310	-0.1051	0.0024	2.8335
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0616	1.4447	-0.0072	-0.0248	-0.0039	1.8215
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1147	0.5230	-0.0030	-0.0086	0.0171	-6.5691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1649	4.0312	-0.0265	-0.0897	0.0026	2.1408
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1649	4.0312	-0.0265	-0.0897	0.0026	2.1408
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0450	1.6160	-0.0086	-0.0295	-0.0022	1.3819
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0848	0.9248	-0.0055	-0.0174	0.0136	-4.9111
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1649	4.0312	-0.0265	-0.0897	0.0026	2.1408
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1649	4.0312	-0.0265	-0.0897	0.0026	2.1408
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0450	1.6160	-0.0086	-0.0295	-0.0022	1.3819
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0848	0.9248	-0.0055	-0.0174	0.0136	-4.9111
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2164	3.8130	-0.0259	-0.0876	0.0012	2.8083
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2164	3.8130	-0.0259	-0.0876	0.0012	2.8083
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0634	0.5927	-0.0020	-0.0074	-0.0051	1.7964
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1165	-0.3290	0.0022	0.0088	0.0158	-6.5943

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.7811
Shear X	-0.3613
Shear Z	-0.0458
Moment X	-0.1555
Moment Y (Twist)	0.0279
Moment Z	11.2299

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.6650
Shear X	-0.2183
Shear Z	-0.0310
Moment X	-0.1051
Moment Y (Twist)	0.0171
Moment Z	6.5943

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	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 2. D + L	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 3. D + (S or Lr or R)	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 3. D + (S or Lr or R)	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 5b. D + 0.7E	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0009	2.0369	0.0001	-0.0002	-0.0001	0.0188
ULS: 8. 0.6D + 0.7E	0.0005	1.2221	0.0000	-0.0001	-0.0001	0.0113
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2087	4.4392	0.0002	-0.0004	-0.0002	2.7861
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2087	4.4392	0.0002	-0.0004	-0.0002	2.7861
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0600	1.3898	0.0001	-0.0001	-0.0002	1.8599
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1291	0.5092	0.0000	-0.0001	0.0002	-6.7710

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1563	3.8386	0.0002	-0.0003	-0.0002	2.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1563	3.8386	0.0002	-0.0003	-0.0002	2.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0452	1.5516	0.0001	-0.0001	-0.0001	1.3996
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0970	0.8911	0.0000	-0.0001	0.0001	-5.0736
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1563	3.8386	0.0002	-0.0003	-0.0002	2.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1563	3.8386	0.0002	-0.0003	-0.0002	2.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0452	1.5516	0.0001	-0.0001	-0.0001	1.3996
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0970	0.8911	0.0000	-0.0001	0.0001	-5.0736
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2091	3.6244	0.0002	-0.0003	-0.0002	2.7786
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2091	3.6244	0.0002	-0.0003	-0.0002	2.7786
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0596	0.5750	0.0000	-0.0000	-0.0001	1.8524
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1287	-0.3055	-0.0000	-0.0000	0.0002	-6.7786

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.4481
Shear X	-0.3494
Shear Z	0.0003
Moment X	-0.0006
Moment Y (Twist)	0.0005
Moment Z	11.5537

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.4392
Shear X	-0.2091
Shear Z	0.0002
Moment X	-0.0004
Moment Y (Twist)	0.0002
Moment Z	6.7786

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	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 2. D + L	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 3. D + (S or Lr or R)	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 3. D + (S or Lr or R)	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 5b. D + 0.7E	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0046	2.1299	0.0130	0.0429	-0.0030	0.0630
ULS: 8. 0.6D + 0.7E	-0.0028	1.2779	0.0078	0.0257	-0.0018	0.0378
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2183	4.6647	0.0311	0.1034	-0.0026	2.8336
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2183	4.6647	0.0311	0.1034	-0.0026	2.8336
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0616	1.4446	0.0072	0.0244	0.0039	1.8226
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1145	0.5230	0.0030	0.0085	-0.0166	-6.5709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1649	4.0310	0.0266	0.0883	-0.0027	2.1409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1649	4.0310	0.0266	0.0883	-0.0027	2.1409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0450	1.6160	0.0087	0.0290	0.0021	1.3827
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0847	0.9247	0.0055	0.0171	-0.0132	-4.9124
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1649	4.0310	0.0266	0.0883	-0.0027	2.1409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1649	4.0310	0.0266	0.0883	-0.0027	2.1409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0450	1.6160	0.0087	0.0290	0.0021	1.3827
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0847	0.9247	0.0055	0.0171	-0.0132	-4.9124

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2164	3.8127	0.0259	0.0862	-0.0014	2.8083
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2164	3.8127	0.0259	0.0862	-0.0014	2.8083
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0635	0.5927	0.0020	0.0072	0.0051	1.7974
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1164	-0.3290	-0.0022	-0.0086	-0.0154	-6.5961

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.7807
Shear X	-0.3612
Shear Z	0.0459
Moment X	0.1530
Moment Y (Twist)	0.0270
Moment Z	11.2329

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.6647
Shear X	-0.2183
Shear Z	0.0311
Moment X	0.1034
Moment Y (Twist)	0.0166
Moment Z	6.5961

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 2. D + L	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 3. D + (S or Lr or R)	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 3. D + (S or Lr or R)	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 5b. D + 0.7E	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0042	1.6431	-0.0810	-0.2647	0.0130	-0.0190
ULS: 8. 0.6D + 0.7E	0.0025	0.9858	-0.0486	-0.1588	0.0078	-0.0114
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1652	3.4835	-0.1982	-0.6461	0.0515	2.2005
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1652	3.4835	-0.1982	-0.6461	0.0515	2.2005
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0401	1.1472	-0.0514	-0.1680	0.0095	1.5218
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1307	0.4733	-0.0033	-0.0136	-0.0245	-5.6191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1229	3.0234	-0.1689	-0.5507	0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1229	3.0234	-0.1689	-0.5507	0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0311	1.2711	-0.0588	-0.1922	0.0104	1.1366
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0990	0.7657	-0.0227	-0.0764	-0.0151	-4.2191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1229	3.0234	-0.1689	-0.5507	0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1229	3.0234	-0.1689	-0.5507	0.0419	1.6456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0311	1.2711	-0.0588	-0.1922	0.0104	1.1366
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0990	0.7657	-0.0227	-0.0764	-0.0151	-4.2191
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1669	2.8263	-0.1658	-0.5402	0.0463	2.2081
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1669	2.8263	-0.1658	-0.5402	0.0463	2.2081
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0384	0.4899	-0.0190	-0.0622	0.0043	1.5294
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1290	-0.1840	0.0291	0.0923	-0.0297	-5.6115

Worst Case Reactions LRFD

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

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.0390
Shear X	-0.2824
Shear Z	-0.2931
Moment X	-0.9567
Moment Y (Twist)	0.0802
Moment Z	9.5431

Result	Value (kip, kip-ft)
Axial	3.4835
Shear X	-0.1669
Shear Z	-0.1982
Moment X	-0.6461
Moment Y (Twist)	0.0515
Moment Z	5.6191

Project Details

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

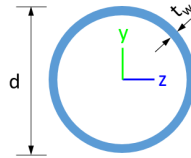


Design Input Information

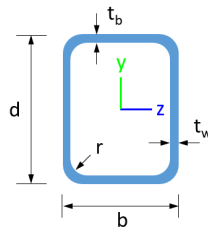
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

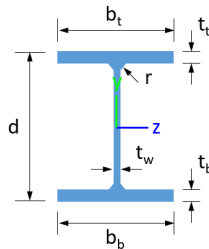
Section Dimensions



ID	Name	d (in)	t_w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties

Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	LS T	LS C	L D
1	7	22.38	22.38	10.66	-	30	20	1
2	4	1.30	1.30	2.00	-	30	20	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.16,1.17,1.17,1.18,1.16,1.18,1.18,1.18,1.12,1.18,1.18,1.1	30	20	1
4	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.44,1.68,1.67,1.67,1.44,1.68,1.68,1.68,1.71,1.68,1.68,1.68,1.7	30	20	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.68,1.66,1.67,1.67,1.68,1.66,1.67,1.67,1.68,1.61,1.67,1.67,1.6	30	20	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.15,1.19,1.19,1.1	30	20	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.63,1.67,1.67,1.6	30	20	1
8	18	1.33	1.33	2.05	1.33,1.33,1.33,1.33,1.33,1.33,1.33,1.32,1.32,1.60,1.60,1.32,1.32,1.60,1.60,1.32,1.32,1.26,1.43,1.32,1.32,1.2	30	20	1
9	1	2.60	2.60	4.00	-	30	20	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.40,1.68,1.67,1.67,1.40,1.68,1.67,1.67,1.69,1.68,1.67,1.67,1.6	30	20	1
11	18	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.35,1.35,1.43,1.51,1.35,1.35,1.43,1.51,1.35,1.35,1.38,2.13,1.35,1.35,1.3	30	20	1
12	4	4.20	4.20	2.00	-	30	20	1
13	18	4.88	4.00	7.50	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.22,1.32,1.19,1.19,1.22,1.32,1.19,1.19,1.20,1.84,1.19,1.19,1.2	30	20	1
14	18	4.88	4.00	7.50	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,2.90,1.42,1.18,1.18,2.90,1.42,1.18,1.18,1.12,1.24,1.18,1.18,1.1	30	20	1
15	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	30	20	1
16	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	30	20	1
101	7	22.38	22.38	10.66	-	30	20	1
102	4	1.30	1.30	2.00	-	30	20	1
103	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.13,1.18,1.18,1.1	30	20	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.20,1.68,1.67,1.67,1.20,1.68,1.67,1.67,1.69,1.68,1.67,1.67,1.6	30	20	1
105	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.62,1.67,1.67,1.6	30	20	1
106	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.15,1.18,1.18,1.1	30	20	1
107	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.63,1.67,1.67,1.6	30	20	1
108	18	1.33	1.33	2.05	2.21,2.21,2.21,2.21,2.21,2.21,2.21,2.28,2.28,1.24,2.06,2.28,2.28,1.24,2.06,2.26,2.26,2.34,2.09,2.26,2.26,2.3	30	20	1
109	1	2.60	2.60	4.00	-	30	20	1
110	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.44,1.68,1.67,1.67,1.44,1.68,1.67,1.67,1.69,1.68,1.67,1.67,1.6	30	20	1
111	18	1.33	1.33	2.05	2.18,2.18,2.18,2.18,2.18,2.18,2.12,2.12,2.09,2.07,2.12,2.12,2.09,2.07,2.14,2.14,2.11,1.55,2.14,2.14,2.1	30	20	1
112	4	1.30	1.30	2.00	-	30	20	1

315	18	10.20	10.20	10.20	1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.21,1.20,1.23,1.23,1.21,1.20,1.23,1.23,1.22,1.16,1.23,1.23,1.2	300	200	1
316	18	10.20	10.20	10.20	1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.10,1.20,1.23,1.23,1.10,1.20,1.23,1.23,1.24,1.21,1.23,1.23,1.2	300	200	1
401	7	22.38	22.38	10.66	-	300	200	1
402	4	4.20	4.20	2.00	-	300	200	1
403	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.19,1.15,1.19,1.19,1.1	300	200	1
404	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.40,1.68,1.67,1.67,1.40,1.68,1.67,1.67,1.69,1.68,1.67,1.67,1.6	300	200	1
405	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.63,1.67,1.67,1.6	300	200	1
406	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.16,1.17,1.17,1.18,1.16,1.18,1.18,1.18,1.12,1.18,1.18,1.1	300	200	1
407	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.68,1.66,1.67,1.67,1.68,1.66,1.67,1.67,1.68,1.61,1.67,1.67,1.6	300	200	1
408	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
409	1	2.60	2.60	4.00	-	300	200	1
410	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.44,1.68,1.67,1.67,1.44,1.68,1.68,1.68,1.71,1.68,1.68,1.68,1.7	300	200	1
411	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
412	4	1.30	1.30	2.00	-	300	200	1
413	18	4.88	4.00	7.50	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.22,1.32,1.19,1.19,1.22,1.32,1.19,1.19,1.20,1.84,1.19,1.19,1.2	300	200	1
414	18	4.88	4.00	7.50	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,2.90,1.42,1.18,1.18,2.90,1.42,1.18,1.18,1.12,1.24,1.18,1.18,1.1	300	200	1
415	18	10.20	10.20	10.20	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.11,1.11,1.10,1.10,1.11,1.11,1.10,1.10,1.11,1.20,1.10,1.10,1.1	300	200	1
416	18	10.20	10.20	10.20	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.54,1.12,1.10,1.10,1.54,1.12,1.10,1.10,1.09,1.11,1.10,1.10,1.0	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	251.16	88.17	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	115.40	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	6.26	29.14	16.61
11	120.60	115.40	23.36	6.45	30.09	45.74
12	142.83	131.65	16.17	16.17	42.85	42.85
13	120.60	84.02	20.89	6.45	30.09	45.74
14	120.60	84.02	19.69	6.45	30.09	45.74
15	120.60	96.18	23.36	6.45	30.09	45.74
16	120.60	96.18	23.36	6.45	30.09	45.74
101	251.16	88.17	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	6.26	29.14	16.61

104	79.65	72.01	10.99	6.26	29.14	16.61
105	79.65	73.44	10.99	6.26	29.14	16.61
106	79.65	74.02	10.99	6.26	29.14	16.61
107	79.65	73.44	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.35	6.45	30.09	45.74
114	120.60	84.03	18.13	6.45	30.09	45.74
115	120.60	33.17	14.95	6.45	30.09	45.74
116	120.60	68.63	15.00	6.45	30.09	45.74
201	251.16	88.17	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	6.26	29.14	16.61
204	79.65	72.01	10.99	6.26	29.14	16.61
205	79.65	73.44	10.99	6.26	29.14	16.61
206	79.65	74.02	10.99	6.26	29.14	16.61
207	79.65	73.44	10.99	6.26	29.14	16.61
208	120.60	115.40	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	6.26	29.14	16.61
211	120.60	115.40	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.02	18.39	6.45	30.09	45.74
214	120.60	84.02	18.33	6.45	30.09	45.74
215	120.60	68.63	15.80	6.45	30.09	45.74
216	120.60	33.17	15.81	6.45	30.09	45.74
301	251.16	88.17	42.30	42.30	75.35	75.35
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	6.26	29.14	16.61
304	79.65	72.01	10.99	6.26	29.14	16.61
305	79.65	73.44	10.99	6.26	29.14	16.61
306	79.65	74.02	10.99	6.26	29.14	16.61
307	79.65	73.44	10.99	6.26	29.14	16.61
308	120.60	115.40	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	6.26	29.14	16.61
311	120.60	115.40	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	84.03	18.34	6.45	30.09	45.74
314	120.60	84.03	18.13	6.45	30.09	45.74
315	120.60	33.17	15.83	6.45	30.09	45.74
316	120.60	33.17	15.08	6.45	30.09	45.74
401	251.16	88.17	42.30	42.30	75.35	75.35
402	142.83	131.65	16.17	16.17	42.85	42.85
403	79.65	74.02	10.99	6.26	29.14	16.61
404	79.65	72.01	10.99	6.26	29.14	16.61
405	79.65	73.44	10.99	6.26	29.14	16.61
406	79.65	74.02	10.99	6.26	29.14	16.61
407	79.65	73.44	10.99	6.26	29.14	16.61

407	79.03	73.44	10.99	0.20	29.14	10.01
408	120.60	96.18	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.01	10.99	6.26	29.14	16.61
411	120.60	96.18	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	84.02	20.89	6.45	30.09	45.74
414	120.60	84.02	19.70	6.45	30.09	45.74
415	120.60	33.17	15.05	6.45	30.09	45.74
416	120.60	33.17	14.93	6.45	30.09	45.74

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	0.057	0.226	0.051	0.004	0.004	0.230	#32	0.598	Not Required	Pass
2	0.001	0.195	0.015	0.046	0.003	0.211	#13	0.034	Not Required	Pass
3	0.001	0.345	0.016	0.034	0.003	0.362	#13	0.044	Not Required	Pass
4	0.000	0.322	0.030	0.033	0.005	0.352	#13	0.117	Not Required	Pass
5	0.001	0.214	0.011	0.035	0.003	0.225	#13	0.073	Not Required	Pass
6	0.001	0.484	0.026	0.050	0.005	0.510	#13	0.044	Not Required	Pass
7	0.001	0.299	0.027	0.049	0.005	0.310	#13	0.073	Not Required	Pass
8	0.001	0.085	0.024	0.027	0.001	0.094	#13	0.088	Not Required	Pass
9	0.001	0.066	0.018	0.003	0.001	0.085	#13	0.198	Not Required	Pass
10	0.001	0.449	0.031	0.046	0.005	0.473	#13	0.078	Not Required	Pass
11	0.001	0.087	0.024	0.029	0.001	0.093	#13	0.088	Not Required	Pass
12	0.001	0.329	0.020	0.066	0.004	0.350	#13	0.167	Not Required	Pass
13	0.002	0.072	0.038	0.038	0.001	0.088	#13	0.265	Not Required	Pass
14	0.002	0.067	0.038	0.036	0.002	0.084	#13	0.177	Not Required	Pass
15	0.000	0.014	0.002	0.011	0.000	0.015	#13	Not Required	Not Required	Pass
16	0.000	0.013	0.002	0.010	0.000	0.014	#13	Not Required	Not Required	Pass
101	0.077	0.265	0.008	0.005	0.001	0.266	#32	0.598	Not Required	Pass
102	0.000	0.375	0.024	0.078	0.004	0.399	#13	0.052	Not Required	Pass
103	0.001	0.574	0.009	0.058	0.001	0.584	#13	0.044	Not Required	Pass
104	0.001	0.545	0.016	0.055	0.003	0.552	#13	0.078	Not Required	Pass
105	0.001	0.356	0.019	0.058	0.004	0.362	#13	0.073	Not Required	Pass
106	0.001	0.555	0.011	0.056	0.003	0.557	#13	0.044	Not Required	Pass
107	0.001	0.344	0.011	0.056	0.002	0.346	#13	0.073	Not Required	Pass
108	0.001	0.037	0.008	0.031	0.001	0.043	#13	0.088	Not Required	Pass
109	0.001	0.065	0.010	0.001	0.000	0.076	#13	0.198	Not Required	Pass
110	0.001	0.521	0.014	0.053	0.002	0.533	#13	0.078	Not Required	Pass
111	0.001	0.037	0.008	0.033	0.001	0.042	#13	0.088	Not Required	Pass
112	0.000	0.355	0.024	0.075	0.004	0.379	#13	0.052	Not Required	Pass
113	0.002	0.165	0.023	0.045	0.001	0.176	#13	0.265	Not Required	Pass
114	0.002	0.164	0.024	0.043	0.001	0.176	#13	0.265	Not Required	Pass
115	0.004	0.185	0.015	0.036	0.001	0.195	#13	0.675	Not Required	Pass
116	0.002	0.175	0.015	0.034	0.001	0.185	#13	0.439	Not Required	Pass
201	0.073	0.273	0.000	0.005	0.000	0.273	#16	0.598	Not Required	Pass
202	0.000	0.346	0.023	0.072	0.004	0.369	#13	0.034	Not Required	Pass
203	0.001	0.539	0.006	0.055	0.001	0.544	#13	0.044	Not Required	Pass
204	0.001	0.503	0.011	0.051	0.002	0.510	#13	0.078	Not Required	Pass
205	0.001	0.334	0.012	0.054	0.002	0.337	#13	0.073	Not Required	Pass

206	0.001	0.539	0.006	0.055	0.001	0.544	#13	0.044	Not Required	Pass
207	0.001	0.334	0.012	0.054	0.002	0.336	#13	0.073	Not Required	Pass
208	0.001	0.040	0.006	0.030	0.001	0.043	#13	0.088	Not Required	Pass
209	0.001	0.059	0.006	0.001	0.000	0.065	#13	0.198	Not Required	Pass
210	0.001	0.503	0.011	0.051	0.002	0.510	#13	0.078	Not Required	Pass
211	0.001	0.043	0.006	0.032	0.001	0.045	#13	0.088	Not Required	Pass
212	0.000	0.346	0.023	0.072	0.004	0.369	#13	0.034	Not Required	Pass
213	0.001	0.149	0.016	0.041	0.001	0.154	#13	0.265	Not Required	Pass
214	0.002	0.143	0.016	0.038	0.001	0.148	#13	0.265	Not Required	Pass
215	0.002	0.140	0.009	0.032	0.001	0.149	#13	0.439	Not Required	Pass
216	0.005	0.131	0.009	0.030	0.001	0.141	#13	0.675	Not Required	Pass
301	0.077	0.266	0.008	0.005	0.001	0.266	#32	0.598	Not Required	Pass
302	0.000	0.355	0.024	0.075	0.004	0.379	#13	0.052	Not Required	Pass
303	0.001	0.556	0.011	0.056	0.003	0.557	#13	0.044	Not Required	Pass
304	0.001	0.522	0.014	0.053	0.002	0.533	#13	0.078	Not Required	Pass
305	0.001	0.345	0.011	0.056	0.002	0.347	#13	0.073	Not Required	Pass
306	0.001	0.575	0.009	0.058	0.001	0.584	#13	0.044	Not Required	Pass
307	0.001	0.356	0.019	0.058	0.004	0.361	#13	0.073	Not Required	Pass
308	0.001	0.046	0.015	0.034	0.001	0.050	#13	0.088	Not Required	Pass
309	0.001	0.065	0.010	0.001	0.000	0.075	#13	0.198	Not Required	Pass
310	0.001	0.545	0.016	0.055	0.003	0.552	#13	0.117	Not Required	Pass
311	0.001	0.052	0.015	0.036	0.001	0.055	#13	0.088	Not Required	Pass
312	0.000	0.375	0.024	0.078	0.004	0.400	#13	0.052	Not Required	Pass
313	0.002	0.165	0.023	0.045	0.001	0.176	#13	0.265	Not Required	Pass
314	0.002	0.164	0.024	0.043	0.001	0.176	#13	0.265	Not Required	Pass
315	0.003	0.134	0.009	0.033	0.001	0.143	#13	0.675	Not Required	Pass
316	0.005	0.124	0.009	0.031	0.001	0.135	#13	0.675	Not Required	Pass
401	0.057	0.226	0.051	0.004	0.004	0.230	#32	0.598	Not Required	Pass
402	0.001	0.329	0.020	0.066	0.004	0.350	#13	0.167	Not Required	Pass
403	0.001	0.484	0.026	0.050	0.005	0.510	#13	0.044	Not Required	Pass
404	0.001	0.449	0.031	0.046	0.005	0.474	#13	0.078	Not Required	Pass
405	0.001	0.299	0.027	0.049	0.005	0.311	#13	0.073	Not Required	Pass
406	0.001	0.345	0.016	0.034	0.003	0.362	#13	0.044	Not Required	Pass
407	0.001	0.214	0.011	0.035	0.003	0.225	#13	0.073	Not Required	Pass
408	0.000	0.013	0.002	0.010	0.000	0.014	#13	Not Required	Not Required	Pass
409	0.001	0.066	0.019	0.003	0.001	0.085	#13	0.198	Not Required	Pass
410	0.000	0.322	0.030	0.033	0.005	0.352	#13	0.117	Not Required	Pass
411	0.000	0.014	0.002	0.011	0.000	0.015	#13	Not Required	Not Required	Pass
412	0.001	0.195	0.015	0.046	0.003	0.211	#13	0.034	Not Required	Pass
413	0.002	0.072	0.038	0.038	0.001	0.088	#13	0.177	Not Required	Pass
414	0.002	0.067	0.038	0.036	0.002	0.084	#13	0.265	Not Required	Pass
415	0.004	0.198	0.024	0.029	0.001	0.207	#13	0.675	Not Required	Pass
416	0.005	0.187	0.024	0.027	0.001	0.199	#13	0.675	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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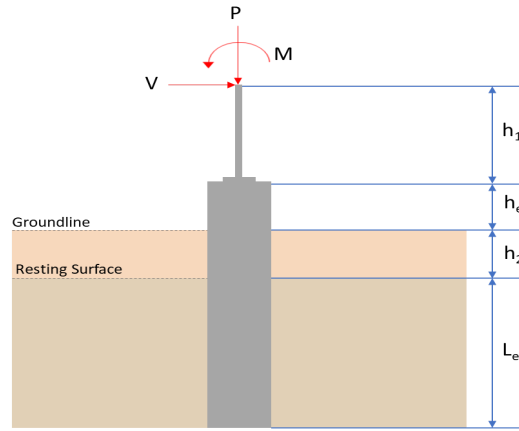
SkyCiv Foundation Design

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

$L = 4.25$ 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)	3.483	5.039
V_x (kip)	-0.167	-0.282
V_z (kip)	0.198	0.293
M_x (kipft)	0.643	0.952
M_z (kipft)	5.618	9.540

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,

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

$$H_o = \frac{(-0.167 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.618 \text{ kipft}) + ((-0.167 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.89459 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.0238 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.198 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.031529 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.643 \text{ kipft}) + ((0.198 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10239 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.3265 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.0238 \text{ ft}), (2.3265 \text{ ft})]$$

$$L_{e,req} = 4.024 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.024 \text{ ft})}{(4.25 \text{ ft})}$$

$$\text{Ratio} = 0.94682$$

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

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

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

$$q = \frac{(3.483 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.21769 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.21769 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.10884$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.026592 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.89459 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.89459 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.89459 \text{ kipft/ft})) + (4 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8608 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.89459 \text{ kipft/ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 \times [(3 \times (0.89459 \text{ kipft/ft})) + (2 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.17728 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.89459 \text{ kipft/ft})) + ((-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.55678 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.8608 \text{ ft})}{2}$$

$$p_a = 0.21456 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.17728 \text{ kip/ft}^2)}{(0.21456 \text{ kip/ft}^2)}$$

$$Ratio = 0.82622$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$$

$$p_s = 0.6375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.55678 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$Ratio = 0.87339$$

Status: **PASS**
Ratio: **0.830**

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

Considering z-direction:

$H_o = 0.031529 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10239 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10239 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0.031529 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.10239 \text{ kipft/ft})) + (4 \times (0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.9984 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.10239 \text{ kipft/ft})) + (3 \times (0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 [(3 \times (0.10239 \text{ kipft/ft})) + (2 \times (0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.047547 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10239 \text{ kipft/ft})) + ((0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.11253 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.9984 \text{ ft})}{2}$$

$$p_a = 0.22488 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.047547 \text{ kip/ft}^2)}{(0.22488 \text{ kip/ft}^2)}$$

$$Ratio = 0.21143$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$$

$$p_s = 0.6375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

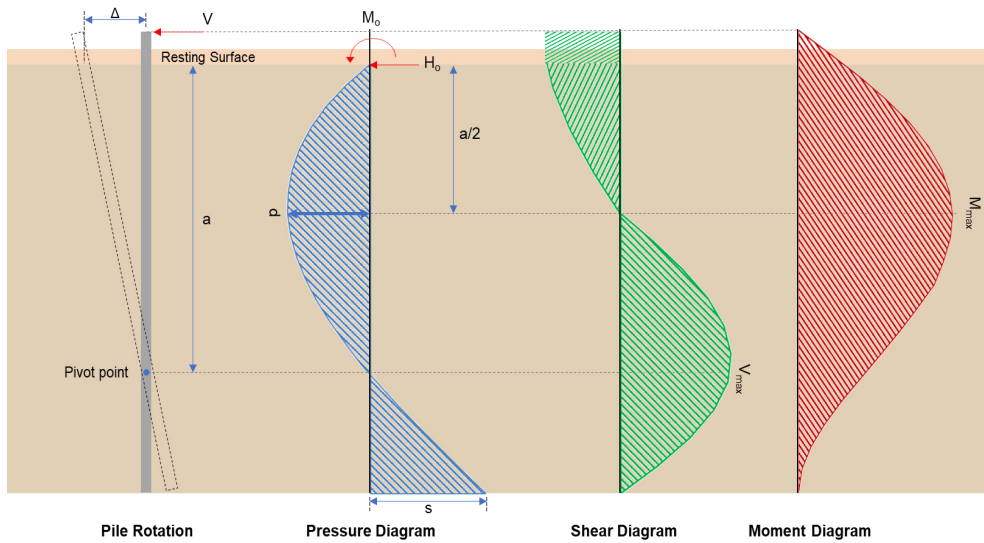
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.11253 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$Ratio = 0.17652$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.282 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044904 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.54 \text{ kipft}) + ((-0.282 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5191 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.5191 \text{ kipft/ft})}{(-0.044904 \text{ kip/ft})}$$

$$E = 33.83 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5191 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.5191 \text{ kipft/ft})) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = \frac{6 \times (1.5191 \text{ kipft/ft}) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft}))}{}$$

$$a = 2.8607 \text{ ft}$$

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

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

$$V_{max} = ((-0.044904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (33.83 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.8607 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (33.83 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.8607 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.6556 \text{ kip}$$

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

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

$$M_{max} = ((-0.044904 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(33.83 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (33.83 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (33.83 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 5.573 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.293 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.046656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.952 \text{ kipft}) + ((0.293 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15159 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15159 \text{ kipft/ft})}{(0.046656 \text{ kip/ft})}$$

$$E = 3.2491 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15159 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0.046656 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.15159 \text{ kipft/ft})) + (4 \times (0.046656 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.9983 \text{ ft}$$

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

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

$$V_{max} = ((0.046656 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2491 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.9983 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2491 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.9983 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37607 \text{ kip}$$

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

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

$$M_{max} = ((0.046656 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(3.2491 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.9983 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2491 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.9983 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2491 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.9983 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.72798 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(5.039 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.429 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.429 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(5.039 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0018836$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.039 \text{ kip} \rightarrow 5039 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5039 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.16 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.16 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.16 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.16 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.53 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.6556 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(2.6556 \text{ kip})}{(110.53 \text{ kip})}$$

$$Ratio = 0.024026$$

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

Considering z-direction:

$V_{max} = 0.37607 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.37607 \text{ kip})}{(110.53 \text{ kip})}$$

$$Ratio = 0.0034023$$

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:

$M_{max} = 5.573 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.573 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.022328$$

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

Considering z-direction:

$M_{max} = 0.72798 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.72798 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0029166$$

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

REFERENCES	CALCULATIONS	RESULTS
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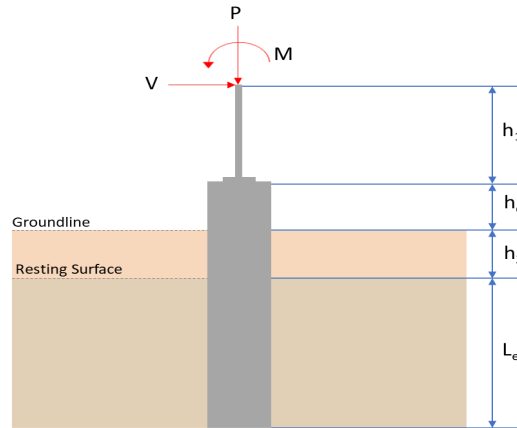
SkyCiv Foundation Design

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

$L = 4.25$ 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)	3.484	5.039
V_x (kip)	-0.167	-0.282
V_z (kip)	-0.198	-0.293
M_x (kipft)	-0.646	-0.957
M_z (kipft)	5.619	9.543

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,

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

$$H_o = \frac{(-0.167 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.619 \text{ kipft}) + ((-0.167 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.89475 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.024 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.198 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.031529 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.646 \text{ kipft}) + ((-0.198 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10287 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7095 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.024 \text{ ft}), (1.7095 \text{ ft})]$$

$$L_{e,req} = 4.024 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.024 \text{ ft})}{(4.25 \text{ ft})}$$

$$\text{Ratio} = 0.94682$$

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

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

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

$$q = \frac{(3.484 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.21775 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.21775 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.10887$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.026592 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.89475 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.89475 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.89475 \text{ kipft/ft})) + (4 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8608 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.89475 \text{ kipft/ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 \times [(3 \times (0.89475 \text{ kipft/ft})) + (2 \times (-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.17731 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.89475 \text{ kipft/ft})) + ((-0.026592 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.55689 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.8608 \text{ ft})}{2}$$

$$p_a = 0.21456 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.17731 \text{ kip/ft}^2)}{(0.21456 \text{ kip/ft}^2)}$$

$$Ratio = 0.82638$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$$

$$p_s = 0.6375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.55689 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$Ratio = 0.87355$$

Status: **PASS**
Ratio: **0.830**

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

Considering z-direction:

$H_o = -0.031529 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10287 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10287 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.031529 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.10287 \text{ kipft/ft})) + (4 \times (-0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.998 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.10287 \text{ kipft/ft})) + (3 \times (-0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 \times [(3 \times (0.10287 \text{ kipft/ft})) + (2 \times (-0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.000091795 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10287 \text{ kipft/ft})) + ((-0.031529 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.023829 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.998 \text{ ft})}{2}$$

$$p_a = 0.22485 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000091795 \text{ kip/ft}^2)}{(0.22485 \text{ kip/ft}^2)}$$

$$Ratio = 0.00040826$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$$

$$p_s = 0.6375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

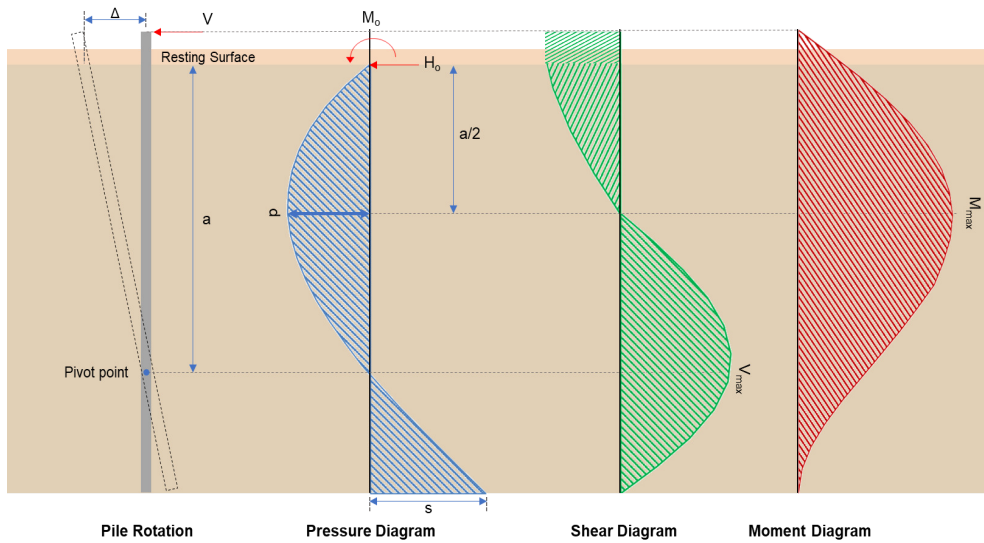
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.023829 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$Ratio = 0.037379$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.282 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044904 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.543 \text{ kipft}) + ((-0.282 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5196 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.5196 \text{ kipft/ft})}{(-0.044904 \text{ kip/ft})}$$

$$E = 33.84 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5196 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.5196 \text{ kipft/ft})) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = \frac{(-0.044904 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (1.5196 \text{ kipft/ft})) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8607 \text{ ft}$$

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

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

$$V_{max} = ((-0.044904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (33.84 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.8607 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (33.84 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.8607 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.6564 \text{ kip}$$

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

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

$$M_{max} = ((-0.044904 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(33.84 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (33.84 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (33.84 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.8607 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 5.5747 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.293 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.957 \text{ kipft}) + ((-0.293 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15239 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15239 \text{ kipft/ft})}{(-0.046656 \text{ kip/ft})}$$

$$E = 3.2662 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15239 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (-0.046656 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.15239 \text{ kipft/ft})) + (4 \times (-0.046656 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.9978 \text{ ft}$$

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

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

$$V_{max} = ((-0.046656 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2662 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.9978 \text{ ft})}{(4.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2662 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.9978 \text{ ft})}{(4.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37739 \text{ kip}$$

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

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

$$M_{max} = ((-0.046656 \text{ kip/ft}) \times (48 \text{ in}) \times (4.25 \text{ ft})) \times \left[\left(\frac{(3.2662 \text{ ft})}{(4.25 \text{ ft})} + \frac{(2.9978 \text{ ft})}{2 \times (4.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2662 \text{ ft})}{(4.25 \text{ ft})} + 3 \right) \times \left(\frac{(2.9978 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2662 \text{ ft})}{(4.25 \text{ ft})} + 2 \right) \times \left(\frac{(2.9978 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.73077 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(5.039 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.429 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.429 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(5.039 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0018836$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.039 \text{ kip} \rightarrow 5039 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5039 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.16 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.16 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.16 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.16 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.53 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.6564 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.6564 \text{ kip})}{(110.53 \text{ kip})}$$

$$Ratio = 0.024033$$

Considering z-direction:

$V_{max} = 0.37739 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.37739 \text{ kip})}{(110.53 \text{ kip})}$$

$$Ratio = 0.0034143$$

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

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:

$M_{max} = 5.5747 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.5747 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.022334$$

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

Considering z-direction:

$M_{max} = 0.73077 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.73077 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0029278$$

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

REFERENCES	CALCULATIONS	RESULTS
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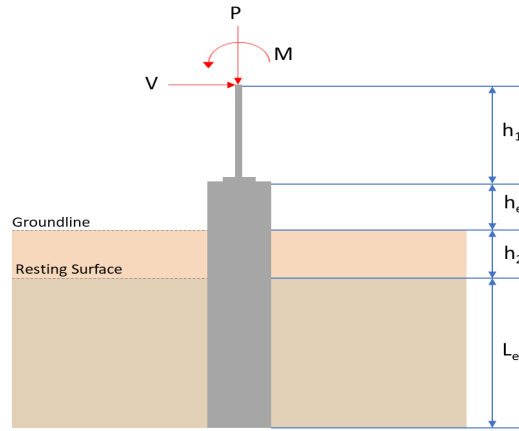
SkyCiv Foundation Design

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

$L = 4.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.665	6.781
V_x (kip)	-0.218	-0.361
V_z (kip)	-0.031	-0.046
M_x (kipft)	-0.105	-0.155
M_z (kipft)	6.594	11.230

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,

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

$$H_o = \frac{(-0.218 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.034713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.594 \text{ kipft}) + ((-0.218 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.05 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2211 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.105 \text{ kipft}) + ((-0.031 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.01672 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0124 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.2211 \text{ ft}), (1.0124 \text{ ft})]$$

$$L_{e,req} = 4.221 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.221 \text{ ft})}{(4.5 \text{ ft})}$$

$$Ratio = 0.938$$

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

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

$$q = \frac{(4.665 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.29156 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.29156 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14578$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.034713 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.05 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.05 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.05 \text{ kipft/ft})) + (4 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0338 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.05 \text{ kipft/ft})) + (3 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.05 \text{ kipft/ft})) + (2 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.18173 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.05 \text{ kipft/ft})) + ((-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.57594 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0338 \text{ ft})}{2}$$

$$p_a = 0.22754 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18173 \text{ kip/ft}^2)}{(0.22754 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79868$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.57594 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.85324$$

Status: **PASS**
Ratio: **0.800**

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

Considering z-direction:

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

$M_o = 0.01672 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.01672 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.01672 \text{ kipft/ft})) + (4 \times (-0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1761 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.01672 \text{ kipft/ft})) + (3 \times (-0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.01672 \text{ kipft/ft})) + (2 \times (-0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 368.6 \times 10^{-9} \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.01672 \text{ kipft/ft})) + ((-0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0033263 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.1761 \text{ ft})}{2}$$

$$p_a = 0.23821 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(368.6 \times 10^{-9} \text{ kip/ft}^2)}{(0.23821 \text{ kip/ft}^2)}$$

$$Ratio = 1.5474 \times 10^{-6}$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

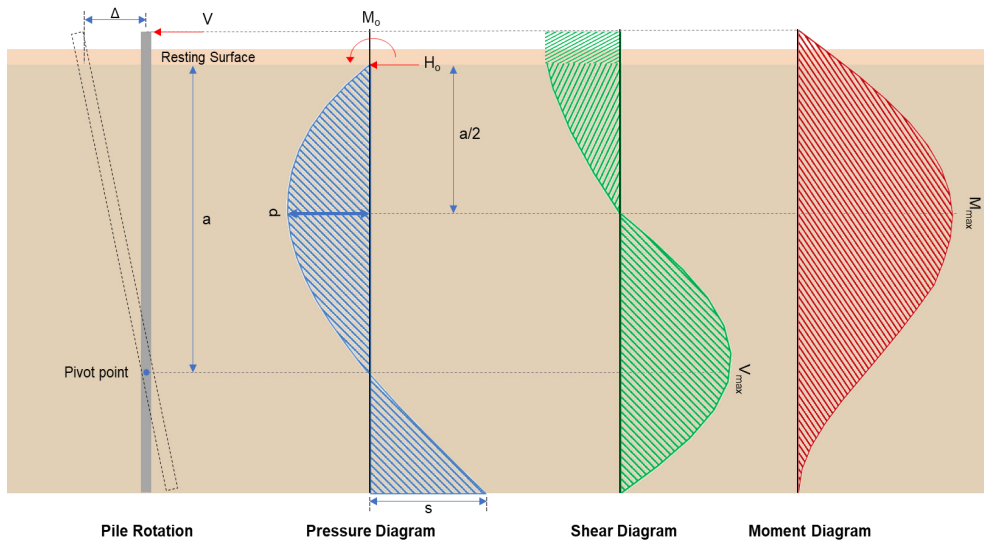
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0033263 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.0049278$$

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,

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

$$H_o = \frac{(-0.361 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.057484 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.23 \text{ kipft}) + ((-0.361 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7882 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.7882 \text{ kipft/ft})}{(-0.057484 \text{ kip/ft})}$$

$$E = 31.108 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7882 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.057484 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.7882 \text{ kipft/ft})) + (4 \times (-0.057484 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.057484 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (1.7882 \text{ kip/ft})) + (4 \times (-0.057484 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.033 \text{ ft}$$

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

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

$$V_{max} = ((-0.057484 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (31.108 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.033 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (31.108 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.033 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.9717 \text{ kip}$$

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

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

$$M_{max} = ((-0.057484 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(31.108 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.033 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (31.108 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.033 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (31.108 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.033 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 6.5912 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.046 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0073248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.155 \text{ kipft}) + ((-0.046 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.024682 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.024682 \text{ kipft/ft})}{(-0.0073248 \text{ kip/ft})}$$

$$E = 3.3696 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.024682 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0073248 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.024682 \text{ kipft/ft})) + (4 \times (-0.0073248 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1766 \text{ ft}$$

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

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

$$V_{max} = ((-0.0073248 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3696 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1766 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3696 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1766 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.058232 \text{ kip}$$

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

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

$$M_{max} = ((-0.0073248 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(3.3696 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1766 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3696 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1766 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3696 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1766 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.11919 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(0.781 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.371 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.371 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.781 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0025348$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.781 \text{ kip} \rightarrow 6781 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6781 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.39 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.39 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.39 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.68 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.9717 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.9717 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.026849$$

Considering z-direction:

$V_{max} = 0.058232 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.058232 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.00052611$$

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

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:

$M_{max} = 6.5912 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.5912 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.026407$$

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

Considering z-direction:

$M_{max} = 0.11919 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.11919 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00047754$$

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

REFERENCES	CALCULATIONS	RESULTS
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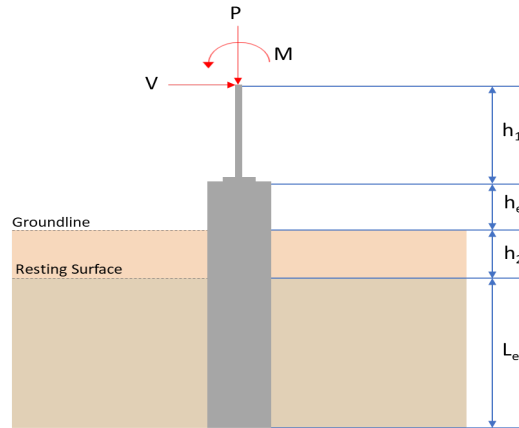
SkyCiv Foundation Design

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

$L = 4.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.439	6.448
V_x (kip)	-0.209	-0.349
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	6.779	11.554

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,

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

$$H_o = \frac{(-0.209 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.03328 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.779 \text{ kipft}) + ((-0.209 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0795 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2696 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.2696 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 4.27 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.27 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.94889$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.439 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.27744 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.27744 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.13872$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.03328 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.0795 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.0795 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.03328 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.0795 \text{ kipft/ft})) + (4 \times (-0.03328 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0317 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.0795 \text{ kipft/ft})) + (3 \times (-0.03328 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.0795 \text{ kipft/ft})) + (2 \times (-0.03328 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.18861 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.0795 \text{ kipft/ft})) + ((-0.03328 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.59531 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0317 \text{ ft})}{2}$$

$$p_a = 0.22738 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18861 \text{ kip/ft}^2)}{(0.22738 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82947$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

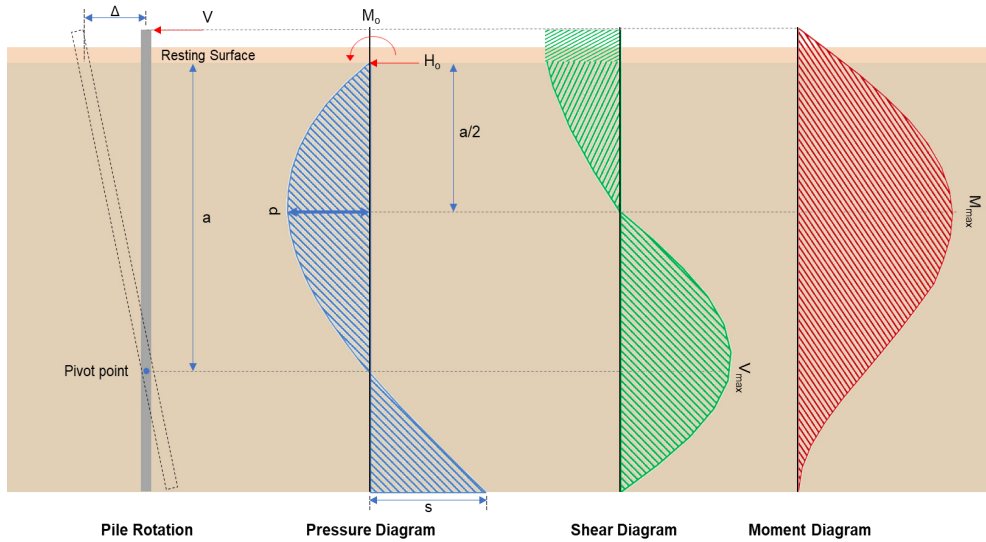
$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.59531 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.830**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.349 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.055573 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.554 \text{ kipft}) + ((-0.349 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.8398 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.8398 \text{ kipft/ft})}{(-0.055573 \text{ kip/ft})}$$

$$E = 33.106 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8398 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.055573 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.8398 \text{ kipft/ft})) + (4 \times (-0.055573 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0312 \text{ ft}$$

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

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

$$V_{max} = ((-0.055573 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (33.106 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0312 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (33.106 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0312 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.0463 \text{ kip}$$

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

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

$$M_{max} = ((-0.055573 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(33.106 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0312 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (33.106 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0312 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (33.106 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0312 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 6.7667 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00015924 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

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

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

$$V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(4.5 \text{ ft})} \right) \times \left(\frac{(3 \text{ ft})}{(4.5 \text{ ft})} - 1 \right) \times \left(\frac{(3 \text{ ft})}{(4.5 \text{ ft})} \right)^2$$

$$V_{max} = 0.00025163 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(3 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left(3 \times \frac{(3 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\frac{(6.448 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.382 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.382 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

22.4.2.2 **Axial Compression Strength (ACI 318-19, LFRD)**

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(6.448 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0024103$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.448 \text{ kip} \rightarrow 6448 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6448 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.35 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.35 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.35 \text{ kip}$$

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

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

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

	<p style="text-align: center;">$V_{s,a} = 737.28 \text{ kip}$</p> <p>$A_v$ - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.66 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.0483 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.0483 \text{ kip})}{(110.66 \text{ kip})}$ $\text{Ratio} = 0.027548$	<p>Status: PASS Ratio: 0.030</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$	

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:

$M_{max} = 6.7667 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.7667 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.02711$$

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

Considering z-direction:

$M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 2.2683 \times 10^{-6}$$

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

REFERENCES	CALCULATIONS	RESULTS
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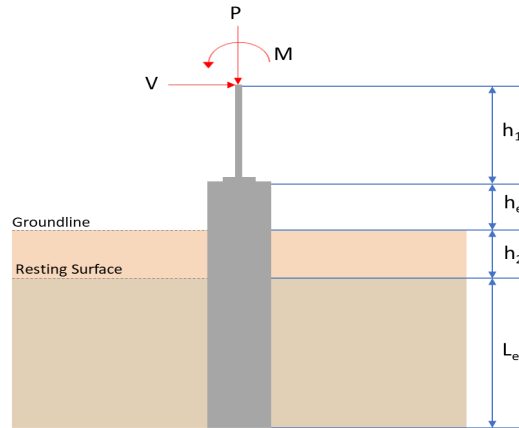
SkyCiv Foundation Design

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

$L = 4.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.665	6.781
V_x (kip)	-0.218	-0.361
V_z (kip)	0.031	0.046
M_x (kipft)	0.103	0.153
M_z (kipft)	6.596	11.233

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,

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

$$H_o = \frac{(-0.218 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.034713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.596 \text{ kipft}) + ((-0.218 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0503 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2215 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.103 \text{ kipft}) + ((0.031 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016401 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1848 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.2215 \text{ ft}), (1.1848 \text{ ft})]$$

$$L_{e,req} = 4.222 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.222 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.93822$$

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

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

$$q = \frac{(4.665 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.29156 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.29156 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14578$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.034713 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.0503 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.0503 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.0503 \text{ kipft/ft})) + (4 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0338 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.0503 \text{ kipft/ft})) + (3 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.0503 \text{ kipft/ft})) + (2 \times (-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.18179 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.0503 \text{ kipft/ft})) + ((-0.034713 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.57613 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0338 \text{ ft})}{2}$$

$$p_a = 0.22754 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18179 \text{ kip/ft}^2)}{(0.22754 \text{ kip/ft}^2)}$$

$$Ratio = 0.79896$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.57613 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.85352$$

Status: **PASS**
Ratio: **0.800**

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

Considering z-direction:

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

$M_o = 0.016401 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.016401 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.016401 \text{ kipft/ft})) + (4 \times (0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1779 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.016401 \text{ kipft/ft})) + (3 \times (0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (0.016401 \text{ kipft/ft})) + (2 \times (0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.006918 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.016401 \text{ kipft/ft})) + ((0.0049363 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.016301 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.1779 \text{ ft})}{2}$$

$$p_a = 0.23835 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.006918 \text{ kip/ft}^2)}{(0.23835 \text{ kip/ft}^2)}$$

$$Ratio = 0.029025$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

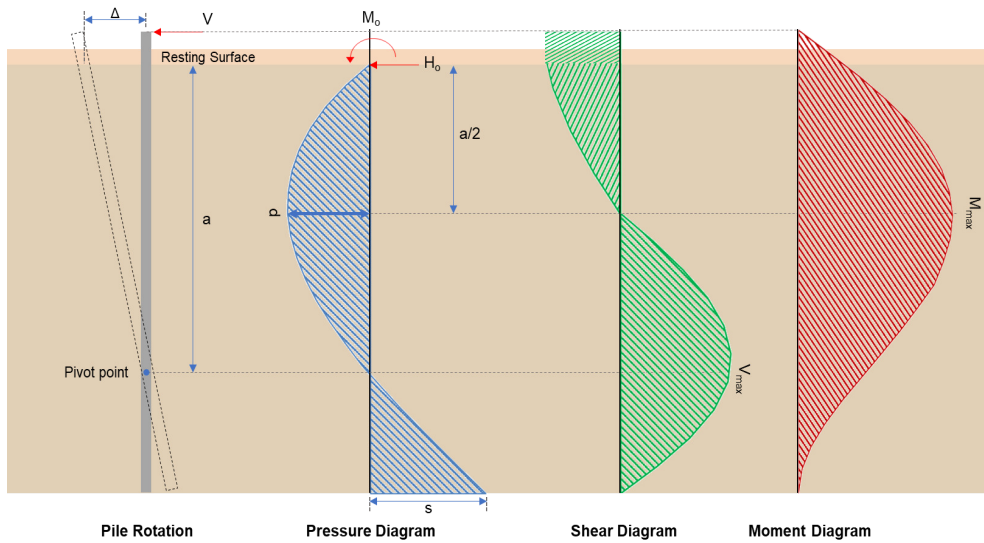
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.016301 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.02415$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.361 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.057484 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(11.233 \text{ kipft}) + ((-0.361 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7887 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(1.7887 \text{ kipft/ft})}{(-0.057484 \text{ kip/ft})}$$

$$E = 31.116 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7887 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.057484 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times 1.7887) + (4 \times (-0.057484) \times 4.5)}$$

$$a = \frac{6 \times (1.7887 \text{ kipft/ft}) + (4 \times (-0.057484 \text{ kip/ft}) \times (4.5 \text{ ft}))}{}$$

$$a = 3.033 \text{ ft}$$

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

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

$$V_{max} = ((-0.057484 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (31.116 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.033 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (31.116 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.033 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.9725 \text{ kip}$$

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

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

$$M_{max} = ((-0.057484 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(31.116 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.033 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (31.116 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.033 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (31.116 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.033 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 6.5929 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.046 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0073248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.153 \text{ kipft}) + ((0.046 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.024363 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.024363 \text{ kipft/ft})}{(0.0073248 \text{ kip/ft})}$$

$$E = 3.3261 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.024363 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0073248 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.024363 \text{ kipft/ft})) + (4 \times (0.0073248 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1778 \text{ ft}$$

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

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

$$V_{max} = ((0.0073248 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3261 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1778 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3261 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1778 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.057735 \text{ kip}$$

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

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

$$M_{max} = ((0.0073248 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(3.3261 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1778 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3261 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1778 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3261 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1778 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.11808 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.781 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.371 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.371 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(6.781 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0025348$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.781 \text{ kip} \rightarrow 6781 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6781 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.39 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.39 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.39 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.68 \text{ kip}$$

Considering x-direction:

$V_{max} = 2.9725 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.9725 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.026856$$

Considering z-direction:

$V_{max} = 0.057735 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.057735 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.00052162$$

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

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:

$M_{max} = 6.5929 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.5929 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.026414$$

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

Considering z-direction:

$M_{max} = 0.11808 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.11808 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00047307$$

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