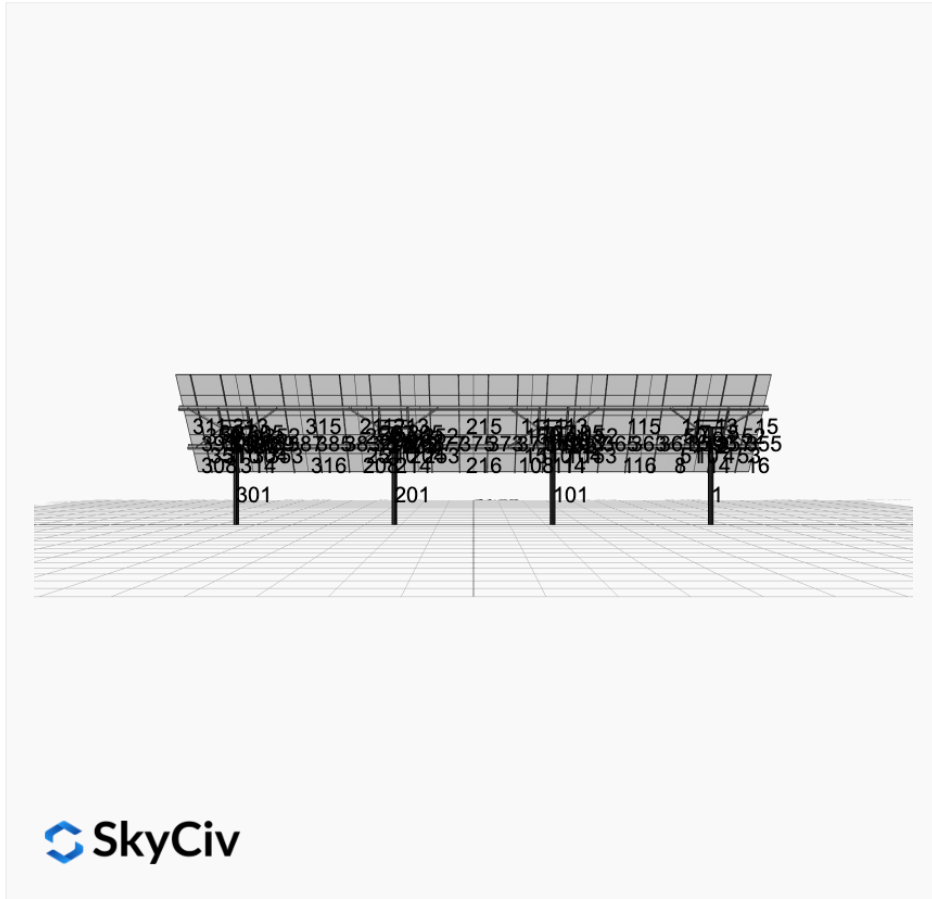


Project Name: Poverty Hill Farms - V1Jb
Location: 13646 W County House Rd, Albion, NY 14411, USA
Unique ID: 4P-22.5-8TOP-HD-45-L-5Hx10W-STRUTS-GB38
Dealer: _____

Date: Tue Nov 26 2024
Number of Modules: 50
Number of Poles: 4
Date Sold: _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	81.58 ft
Winter Tilt Angle	46
Front Edge Clearance	7 ft

MT Solar Bill of Materials (4P-22.5-8TOP-HD-45-L-5Hx10W-STRUTS-GB38)

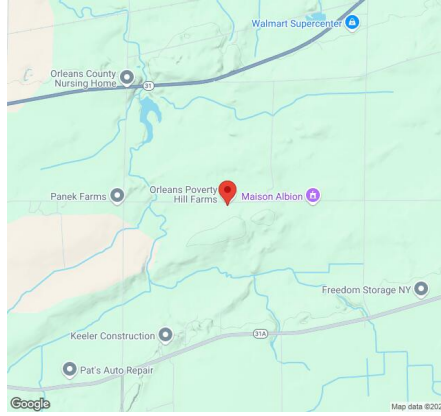
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	12
MTS-CLAMP-ANGLE-4PK	Angle Clamp	10

Rail Bill of Materials

Part	Qty
Rails (223in)	20
Rail Attachment	80

Part	Qty
Module Mid Clamp	80
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: 13646 W County House Rd, Albion, NY 14411, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	96.90in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Winter Tilt Angle:	46
Front Edge Clearance:	7
Total Array Height at Tilt:	20.52 ft
Total Frame Length:	82.50 ft
Frame Weight:	6241 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	81.58 ft
Rail Length:	225.50 in
Rail Spacing:	4.08 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	13.76 ft
Number of Poles:	4
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 8.00 ft Pile 3: 8.00 ft Pile 4: 7.50 ft
Foundation Volume:	18.370 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	13646 W County House Rd, Albion, NY 14411, USA
Wind Speed:	102 mph

Snow Load:

50 psf

Design Disclaimer

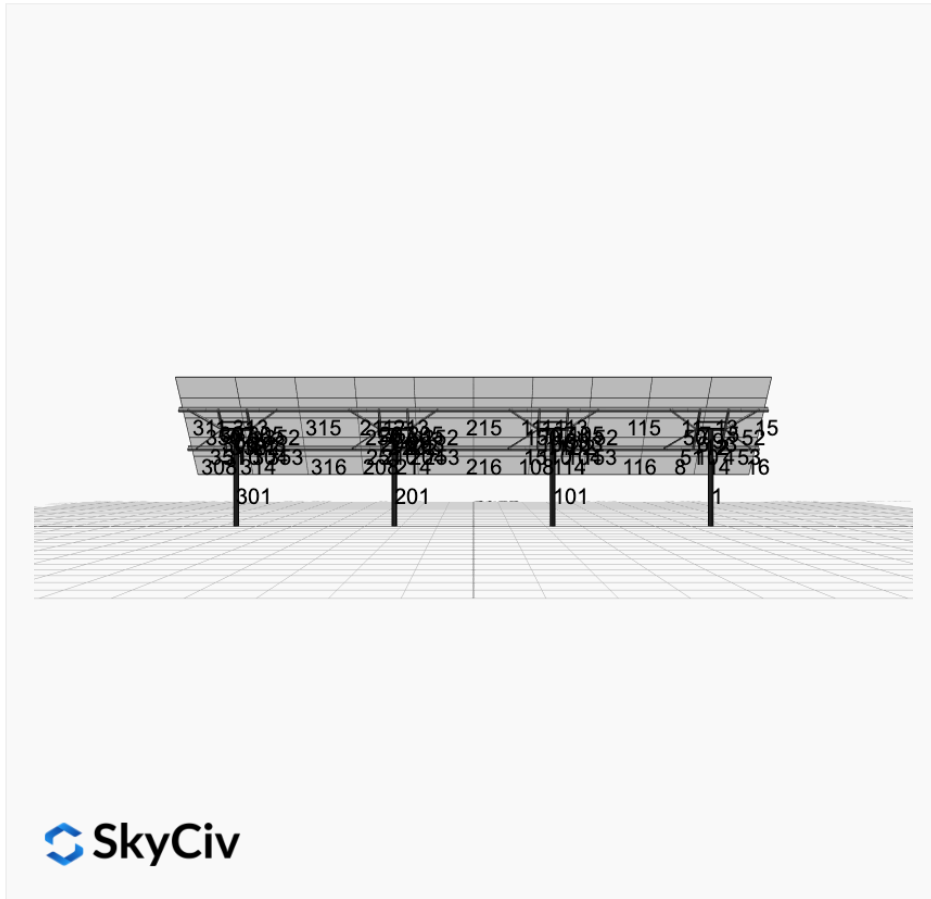
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

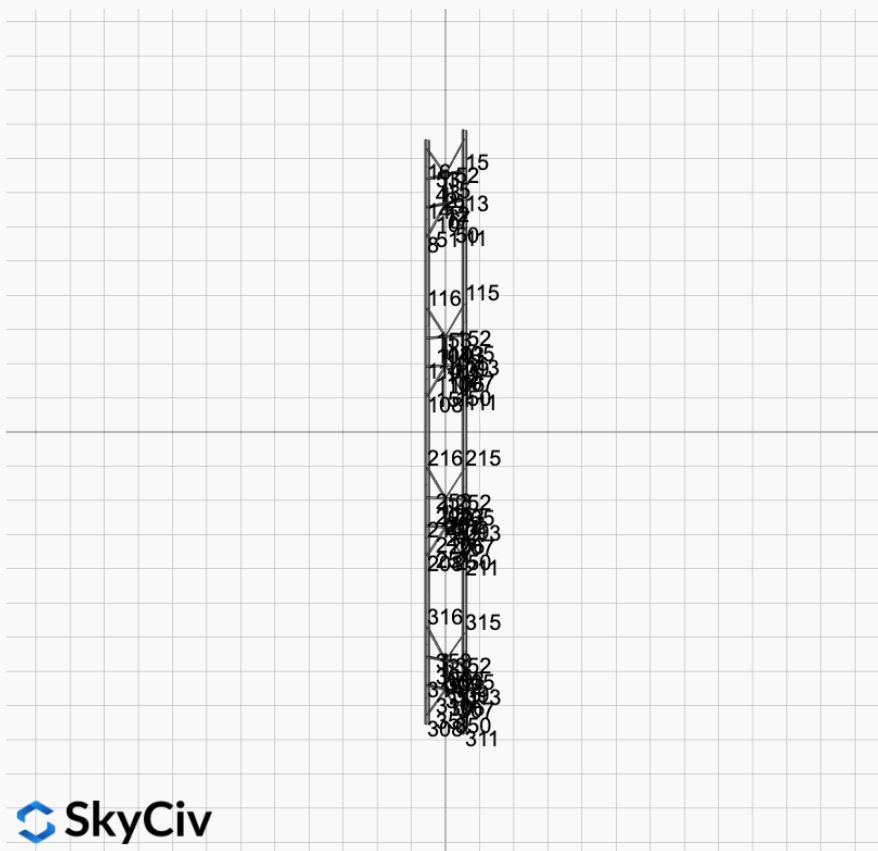
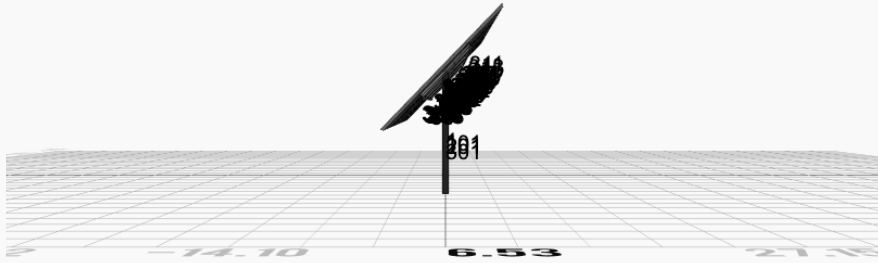
AutoDesigner Input

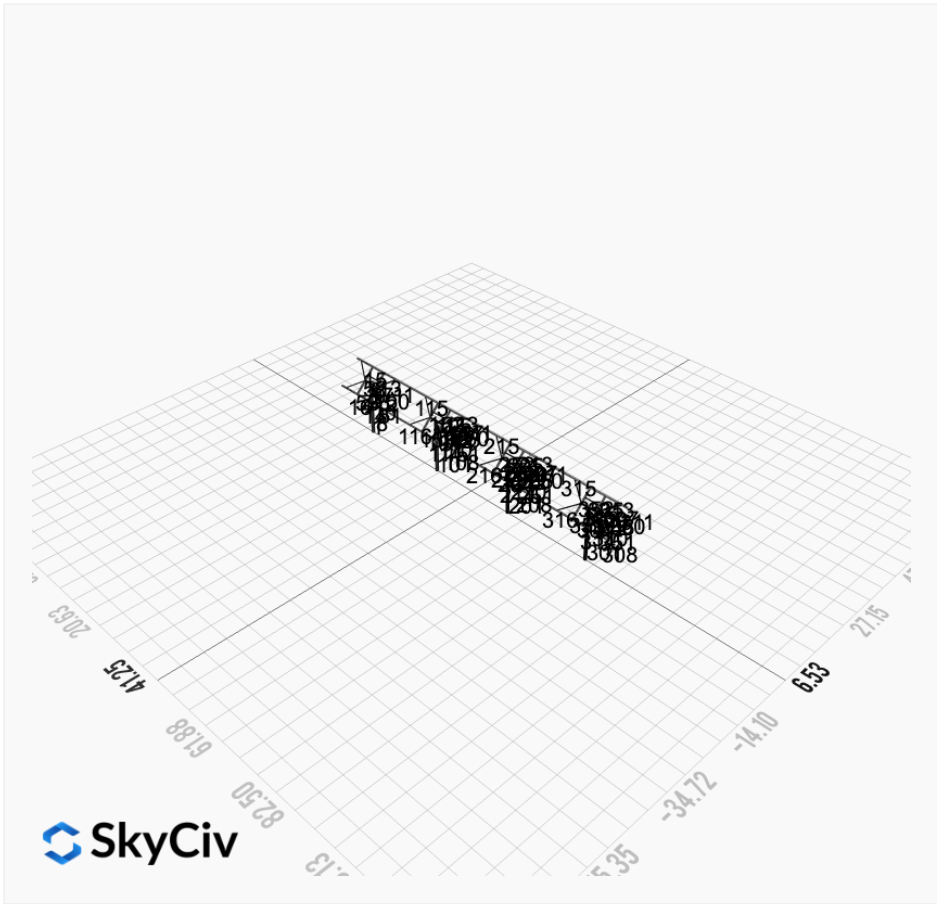
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Design Notes:

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

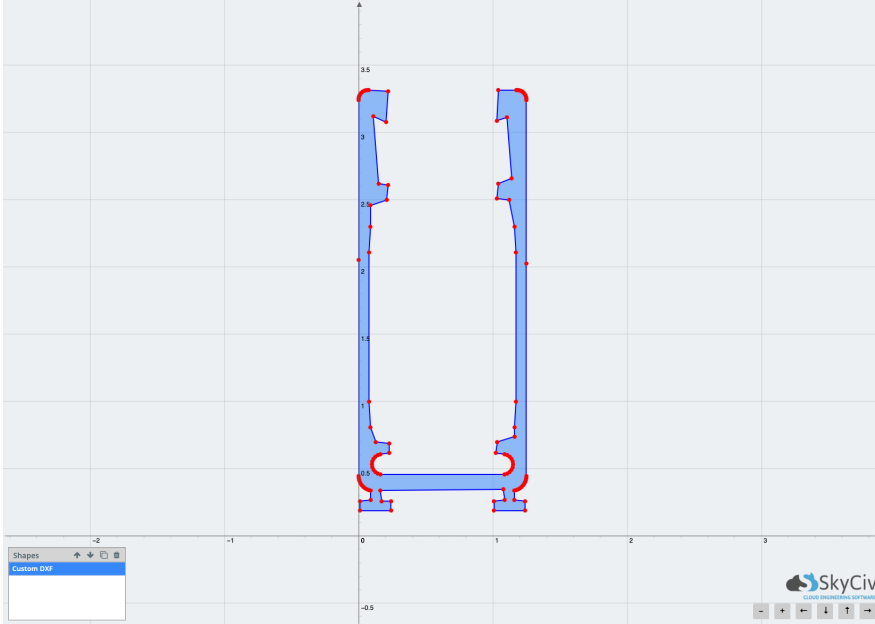






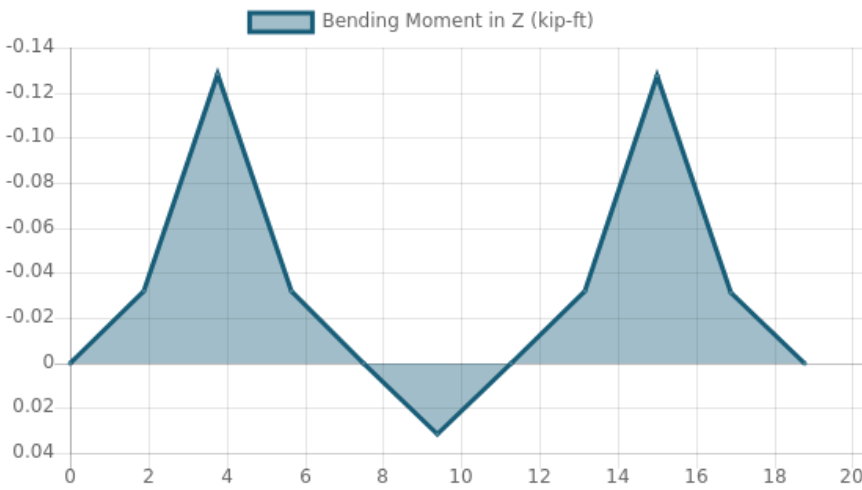
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 4.07916666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0374 kip/ft
Snow (Y): -0.0387 kip/ft
Wind uplift Case A: 0.0849 kip/ft
Wind downforce Case A: 0.0849 kip/ft
Dead (Panel load) (X): 0.0124 kip/ft
Dead (Panel load) (Y): -0.0129 kip/ft

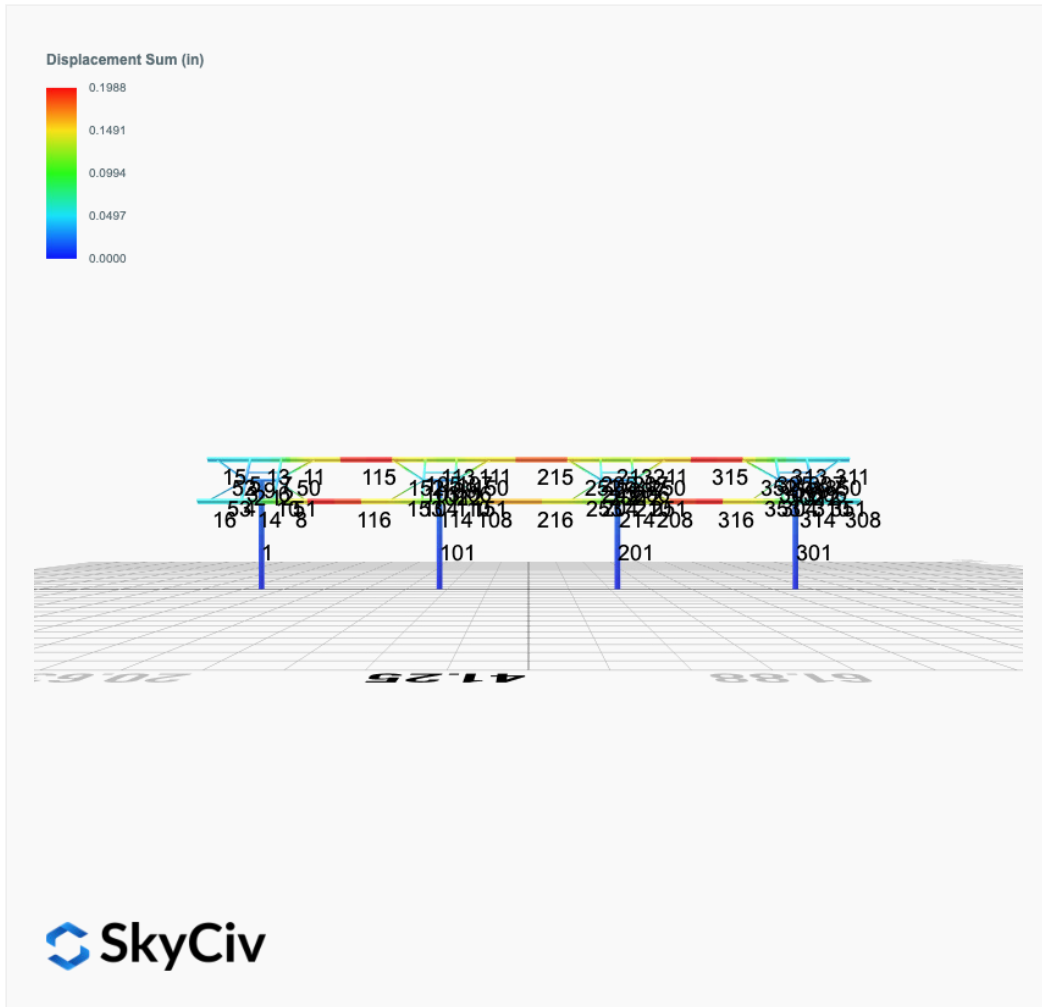


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	18.55283932	0.538	PASS
Material Yield	34.5	18.55283932	0.538	PASS
Material Strength	37	18.55283932	0.501	PASS

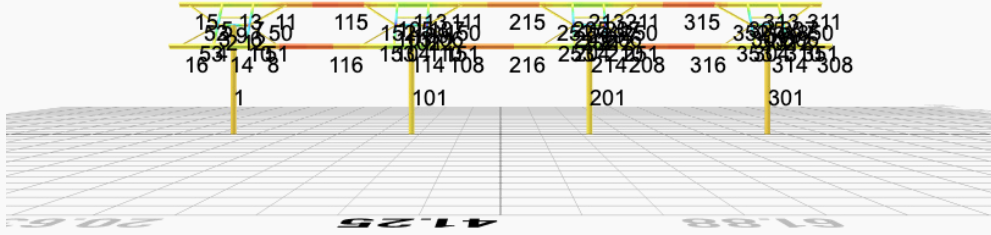
Member 1, ULS: 1. 1.4D



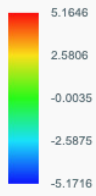
FEM Results (Envelope Worst Case for each member)



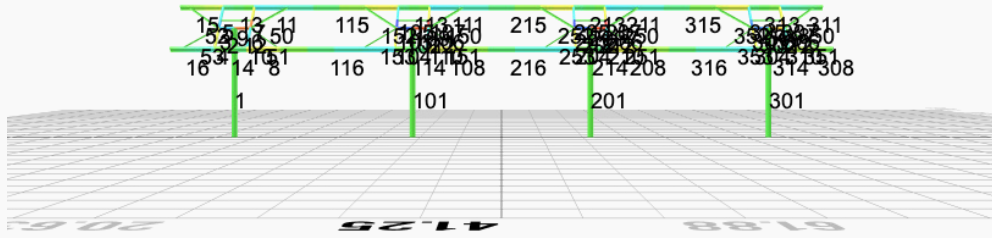
Top Bending Stress Z (ksi)



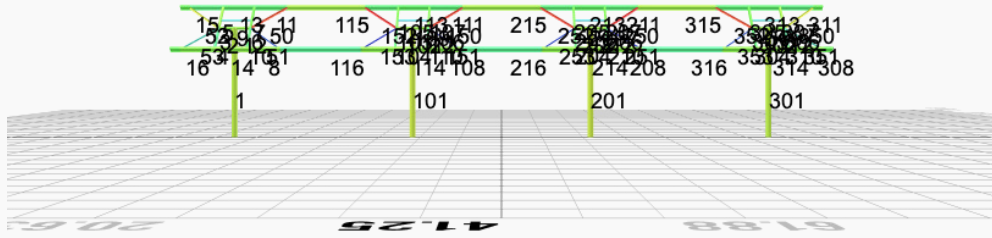
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 2. D + L	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 3. D + (S or Lr or R)	0.0523	5.9441	0.1464	0.6012	-0.3674	-0.6558
ULS: 3. D + (S or Lr or R)	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0440	5.1483	0.1232	0.5061	-0.3090	-0.5497
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 5b. D + 0.7E	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0440	5.1483	0.1232	0.5061	-0.3090	-0.5497
ULS: 8. 0.6D + 0.7E	0.0115	1.6566	0.0323	0.1325	-0.0802	-0.1388
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1314	5.6764	0.2320	0.9362	-1.1258	43.9027
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1607	-0.1522	-0.1208	-0.4797	0.8375	-42.9800
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3190	7.3349	0.2569	1.0426	-1.0531	32.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0440	5.1483	0.1232	0.5061	-0.3090	-0.5497
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4001	2.9634	-0.0077	-0.0193	0.4194	-32.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0440	5.1483	0.1232	0.5061	-0.3090	-0.5497
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3438	4.9475	0.1874	0.7573	-0.8778	32.8692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3753	0.5761	-0.0772	-0.3046	0.5947	-32.2928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0192	2.7609	0.0538	0.2208	-0.1337	-0.2313
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1391	4.5720	0.2105	0.8479	-1.0723	43.9952
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0115	1.6566	0.0323	0.1325	-0.0802	-0.1388
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1530	-1.2566	-0.1424	-0.5680	0.8910	-42.8875
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0115	1.6566	0.0323	0.1325	-0.0802	-0.1388

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	10.8328
Shear X	-5.2796
Shear Z	0.4131
Moment X	1.6713
Moment Y (Twist)	1.9572
Moment Z	74.3385

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	7.3349
Shear X	-3.1607
Shear Z	0.2569
Moment X	1.0426
Moment Y (Twist)	1.1258
Moment Z	43.9952

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	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 2. D + L	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 3. D + (S or Lr or R)	-0.0523	7.1186	-0.0116	-0.0461	0.0579	0.7528
ULS: 3. D + (S or Lr or R)	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0440	6.1380	-0.0097	-0.0388	0.0487	0.6351

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 5b. D + 0.7E	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0440	6.1380	-0.0097	-0.0388	0.0487	0.6351
ULS: 8. 0.6D + 0.7E	-0.0115	1.9178	-0.0025	-0.0102	0.0127	0.1693
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7557	6.9316	0.0073	0.0319	-0.0854	52.5079
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7264	-0.5413	-0.0140	-0.0597	0.1154	-50.1879
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8464	8.9395	-0.0011	-0.0022	-0.0312	39.8044
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0440	6.1380	-0.0097	-0.0388	0.0487	0.6351
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7652	3.3348	-0.0171	-0.0709	0.1194	-37.2174
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0440	6.1380	-0.0097	-0.0388	0.0487	0.6351
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8216	5.9978	0.0044	0.0197	-0.0588	39.4514
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7900	0.3931	-0.0116	-0.0490	0.0918	-37.5703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0192	3.1963	-0.0042	-0.0169	0.0212	0.2822
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7480	5.6531	0.0090	0.0387	-0.0939	52.3950
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0115	1.9178	-0.0025	-0.0102	0.0127	0.1693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7341	-1.8198	-0.0123	-0.0529	0.1069	-50.3008
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0115	1.9178	-0.0025	-0.0102	0.0127	0.1693

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2268
Shear X	-6.2618
Shear Z	-0.0267
Moment X	-0.1136
Moment Y (Twist)	0.2101
Moment Z	89.2539

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.9395
Shear X	-3.7557
Shear Z	-0.0171
Moment X	-0.0709
Moment Y (Twist)	0.1194
Moment Z	52.5079

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	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 2. D + L	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 3. D + (S or Lr or R)	-0.0523	7.1186	0.0116	0.0461	-0.0579	0.7528
ULS: 3. D + (S or Lr or R)	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0440	6.1380	0.0097	0.0388	-0.0487	0.6351
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 5b. D + 0.7E	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0440	6.1380	0.0097	0.0388	-0.0487	0.6351
ULS: 8. 0.6D + 0.7E	-0.0115	1.9178	0.0025	0.0102	-0.0127	0.1693
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7557	6.9316	-0.0073	-0.0319	0.0855	52.5079
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7264	-0.5413	0.0140	0.0597	-0.1154	-50.1879
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8464	8.9395	0.0011	0.0022	0.0313	39.8044
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0440	6.1380	0.0097	0.0388	-0.0487	0.6351
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7652	3.3348	0.0171	0.0709	-0.1194	-37.2174
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0440	6.1380	0.0097	0.0388	-0.0487	0.6351
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8216	5.9978	-0.0044	-0.0197	0.0588	39.4514
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7900	0.3931	0.0116	0.0490	-0.0918	-37.5703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0192	3.1963	0.0042	0.0169	-0.0212	0.2822
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7480	5.6531	-0.0090	-0.0386	0.0939	52.3950
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0115	1.9178	0.0025	0.0102	-0.0127	0.1693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7341	-1.8198	0.0123	0.0529	-0.1069	-50.3008
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0115	1.9178	0.0025	0.0102	-0.0127	0.1693

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2268
Shear X	-6.2618
Shear Z	0.0267
Moment X	0.1143
Moment Y (Twist)	0.2105
Moment Z	89.2540

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.9395
Shear X	-3.7557
Shear Z	0.0171
Moment X	0.0709
Moment Y (Twist)	0.1194
Moment Z	52.5079

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	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 2. D + L	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 3. D + (S or Lr or R)	0.0523	5.9441	-0.1464	-0.6011	0.3674	-0.6558
ULS: 3. D + (S or Lr or R)	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0440	5.1483	-0.1232	-0.5060	0.3090	-0.5497
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 5b. D + 0.7E	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0440	5.1483	-0.1232	-0.5060	0.3090	-0.5497
ULS: 8. 0.6D + 0.7E	0.0115	1.6566	-0.0323	-0.1325	0.0802	-0.1388
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1314	5.6764	-0.2320	-0.9362	1.1258	43.9027
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.1607	-0.1522	0.1208	0.4797	-0.8375	-42.9800
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3190	7.3349	-0.2569	-1.0426	1.0531	32.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0440	5.1483	-0.1232	-0.5060	0.3090	-0.5497
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4001	2.9634	0.0077	0.0193	-0.4194	-32.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0440	5.1483	-0.1232	-0.5060	0.3090	-0.5497
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3438	4.9476	-0.1874	-0.7573	0.8778	32.8692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.3753	0.5761	0.0772	0.3046	-0.5947	-32.2928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0192	2.7609	-0.0538	-0.2208	0.1337	-0.2313

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1391	4.5720	-0.2105	-0.8479	1.0724	43.9952
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0115	1.6566	-0.0323	-0.1325	0.0802	-0.1388
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.1530	-1.2566	0.1424	0.5680	-0.8910	-42.8875
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0115	1.6566	-0.0323	-0.1325	0.0802	-0.1388

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.8329
Shear X	-5.2795
Shear Z	-0.4131
Moment X	-1.6717
Moment Y (Twist)	1.9577
Moment Z	74.3390

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.3349
Shear X	-3.1607
Shear Z	-0.2569
Moment X	-1.0426
Moment Y (Twist)	1.1258
Moment Z	43.9952

Project Details

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

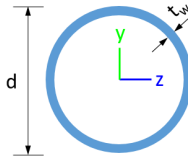


Design Input Information

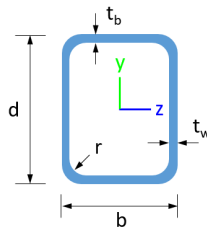
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

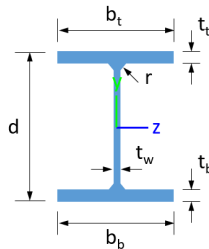
Section Dimensions



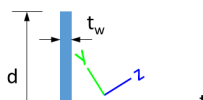
ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



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



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



10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	24.08	6.12	40.24	43.62
14	133.20	85.85	24.12	6.12	40.24	43.62
15	133.20	107.59	32.87	6.12	40.24	43.62
16	133.20	107.59	32.87	6.12	40.24	43.62
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15
101	574.32	198.66	123.94	123.94	172.30	172.30
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.63	6.12	40.24	43.62
114	133.20	85.85	23.64	6.12	40.24	43.62
115	133.20	19.55	12.15	6.12	40.24	43.62
116	133.20	19.55	12.68	6.12	40.24	43.62
150	41.27	8.45	1.63	0.88	15.23	10.15
151	41.27	8.45	1.63	0.88	15.23	10.15
152	41.27	8.45	1.63	0.88	15.23	10.15
153	41.27	8.45	1.63	0.88	15.23	10.15
201	574.32	198.66	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.64	6.12	40.24	43.62
214	133.20	85.85	23.64	6.12	40.24	43.62
215	133.20	19.55	12.28	6.12	40.24	43.62
216	133.20	19.55	12.56	6.12	40.24	43.62
250	41.27	8.45	1.63	0.88	15.23	10.15
251	41.27	8.45	1.63	0.88	15.23	10.15
252	41.27	8.45	1.63	0.88	15.23	10.15
253	41.27	8.45	1.63	0.88	15.23	10.15
301	574.32	198.66	123.94	123.94	172.30	172.30

301	377.32	196.66	123.97	123.97	172.50	172.50
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	107.59	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	107.59	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.09	6.12	40.24	43.62
314	133.20	85.85	24.13	6.12	40.24	43.62
315	133.20	19.55	12.18	6.12	40.24	43.62
316	133.20	19.55	12.27	6.12	40.24	43.62
350	41.27	8.45	1.63	0.88	15.23	10.15
351	41.27	8.45	1.63	0.88	15.23	10.15
352	41.27	8.45	1.63	0.88	15.23	10.15
353	41.27	8.45	1.63	0.88	15.23	10.15

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	0.055	0.600	0.032	0.031	0.002	0.638	#13	0.602	Not Required	Pass
2	0.005	0.284	0.189	0.072	0.038	0.440	#13	0.053	Not Required	Pass
3	0.007	0.499	0.115	0.049	0.049	0.557	#21	0.045	Not Required	Pass
4	0.007	0.500	0.064	0.050	0.011	0.555	#13	0.080	Not Required	Pass
5	0.007	0.310	0.035	0.050	0.010	0.327	#13	0.074	Not Required	Pass
6	0.003	0.696	0.144	0.071	0.055	0.773	#13	0.045	Not Required	Pass
7	0.003	0.431	0.012	0.069	0.004	0.436	#13	0.074	Not Required	Pass
8	0.005	0.091	0.142	0.047	0.012	0.166	#21	0.095	Not Required	Pass
9	0.021	0.069	0.073	0.003	0.003	0.146	#13	0.136	Not Required	Pass
10	0.005	0.670	0.062	0.067	0.008	0.674	#13	0.120	Not Required	Pass
11	0.003	0.082	0.143	0.049	0.012	0.169	#21	0.063	Not Required	Pass
12	0.006	0.445	0.278	0.098	0.051	0.713	#13	0.053	Not Required	Pass
13	0.004	0.201	0.069	0.061	0.009	0.235	#21	0.190	Not Required	Pass
14	0.009	0.192	0.059	0.058	0.009	0.234	#13	0.286	Not Required	Pass
15	0.004	0.058	0.024	0.025	0.006	0.073	#21	0.179	Not Required	Pass
16	0.005	0.058	0.018	0.025	0.006	0.068	#13	0.179	Not Required	Pass
50	0.243	0.009	0.004	0.002	0.001	0.255	#23	0.783	Not Required	Pass
51	0.049	0.005	0.014	0.001	0.002	0.063	#23	0.522	Not Required	Pass
52	0.083	0.009	0.004	0.001	0.000	0.094	#21	0.783	Not Required	Pass
53	0.016	0.005	0.015	0.001	0.001	0.032	#21	0.522	Not Required	Pass
101	0.067	0.720	0.002	0.036	0.000	0.751	#13	0.602	Not Required	Pass
102	0.007	0.456	0.281	0.105	0.052	0.710	#13	0.053	Not Required	Pass
103	0.004	0.722	0.161	0.072	0.065	0.813	#13	0.045	Not Required	Pass
104	0.004	0.734	0.053	0.074	0.006	0.763	#13	0.120	Not Required	Pass
105	0.004	0.447	0.030	0.072	0.012	0.464	#13	0.074	Not Required	Pass
106	0.003	0.738	0.158	0.074	0.065	0.826	#13	0.045	Not Required	Pass
107	0.003	0.458	0.034	0.074	0.012	0.481	#13	0.074	Not Required	Pass

108	0.009	0.074	0.125	0.050	0.011	0.157	#21	0.095	Not Required	Pass
109	0.027	0.059	0.059	0.001	0.000	0.126	#13	0.136	Not Required	Pass
110	0.004	0.732	0.058	0.073	0.007	0.762	#13	0.080	Not Required	Pass
111	0.005	0.062	0.125	0.050	0.011	0.164	#21	0.095	Not Required	Pass
112	0.007	0.453	0.288	0.105	0.053	0.722	#13	0.053	Not Required	Pass
113	0.005	0.265	0.035	0.063	0.007	0.290	#13	0.286	Not Required	Pass
114	0.013	0.293	0.033	0.065	0.007	0.313	#13	0.286	Not Required	Pass
115	0.059	0.493	0.204	0.052	0.017	0.573	#13	0.925	Not Required	Pass
116	0.056	0.481	0.207	0.053	0.018	0.561	#13	0.925	Not Required	Pass
150	0.231	0.009	0.004	0.003	0.002	0.242	#23	0.783	Not Required	Pass
151	0.046	0.005	0.014	0.001	0.002	0.060	#23	0.522	Not Required	Pass
152	0.253	0.009	0.004	0.003	0.002	0.265	#21	0.783	Not Required	Pass
153	0.051	0.005	0.014	0.001	0.002	0.065	#21	0.522	Not Required	Pass
201	0.067	0.720	0.002	0.036	0.000	0.751	#13	0.602	Not Required	Pass
202	0.007	0.453	0.288	0.105	0.053	0.722	#13	0.053	Not Required	Pass
203	0.003	0.738	0.158	0.074	0.065	0.826	#13	0.045	Not Required	Pass
204	0.004	0.732	0.058	0.073	0.007	0.762	#13	0.080	Not Required	Pass
205	0.003	0.458	0.034	0.074	0.012	0.481	#13	0.074	Not Required	Pass
206	0.004	0.722	0.161	0.072	0.065	0.813	#13	0.045	Not Required	Pass
207	0.004	0.447	0.030	0.072	0.012	0.465	#13	0.074	Not Required	Pass
208	0.009	0.065	0.164	0.053	0.013	0.211	#21	0.095	Not Required	Pass
209	0.027	0.059	0.059	0.001	0.000	0.126	#13	0.136	Not Required	Pass
210	0.004	0.734	0.053	0.074	0.006	0.763	#13	0.120	Not Required	Pass
211	0.005	0.067	0.159	0.052	0.013	0.215	#21	0.095	Not Required	Pass
212	0.007	0.456	0.281	0.105	0.052	0.710	#13	0.053	Not Required	Pass
213	0.005	0.265	0.035	0.063	0.007	0.290	#13	0.286	Not Required	Pass
214	0.013	0.293	0.033	0.065	0.007	0.313	#13	0.286	Not Required	Pass
215	0.063	0.414	0.164	0.050	0.016	0.470	#21	0.925	Not Required	Pass
216	0.055	0.374	0.161	0.050	0.016	0.434	#13	0.925	Not Required	Pass
250	0.253	0.009	0.004	0.003	0.002	0.265	#21	0.783	Not Required	Pass
251	0.051	0.005	0.014	0.001	0.002	0.065	#21	0.522	Not Required	Pass
252	0.231	0.009	0.004	0.003	0.002	0.242	#23	0.783	Not Required	Pass
253	0.046	0.005	0.014	0.001	0.002	0.060	#23	0.522	Not Required	Pass
301	0.055	0.600	0.032	0.031	0.002	0.638	#13	0.602	Not Required	Pass
302	0.006	0.445	0.278	0.098	0.051	0.713	#13	0.053	Not Required	Pass
303	0.003	0.696	0.144	0.071	0.055	0.773	#13	0.045	Not Required	Pass
304	0.005	0.670	0.062	0.067	0.008	0.674	#13	0.120	Not Required	Pass
305	0.003	0.431	0.012	0.069	0.004	0.436	#13	0.074	Not Required	Pass
306	0.007	0.499	0.115	0.049	0.049	0.558	#21	0.045	Not Required	Pass
307	0.007	0.310	0.034	0.050	0.010	0.327	#13	0.074	Not Required	Pass
308	0.005	0.058	0.018	0.025	0.006	0.068	#13	0.268	Not Required	Pass
309	0.021	0.069	0.073	0.003	0.003	0.146	#13	0.136	Not Required	Pass
310	0.007	0.500	0.064	0.050	0.011	0.555	#13	0.080	Not Required	Pass
311	0.004	0.058	0.024	0.025	0.006	0.073	#21	0.179	Not Required	Pass
312	0.005	0.284	0.189	0.072	0.038	0.440	#13	0.053	Not Required	Pass
313	0.004	0.201	0.069	0.061	0.009	0.235	#21	0.190	Not Required	Pass
314	0.009	0.192	0.059	0.058	0.009	0.234	#13	0.286	Not Required	Pass
315	0.059	0.500	0.204	0.049	0.017	0.580	#13	0.616	Not Required	Pass
316	0.056	0.499	0.207	0.047	0.018	0.579	#13	0.925	Not Required	Pass
350	0.083	0.009	0.004	0.001	0.000	0.094	#21	0.783	Not Required	Pass
351	0.016	0.005	0.015	0.001	0.001	0.032	#21	0.522	Not Required	Pass
352	0.243	0.009	0.004	0.002	0.001	0.255	#23	0.783	Not Required	Pass

353	0.049	0.005	0.014	0.001	0.002	0.063	#23	0.522	Not Required	Pass
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Definitions

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

REFERENCES	CALCULATIONS	RESULTS
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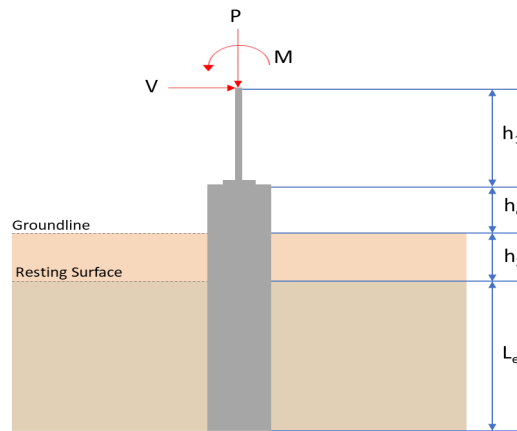
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.5$ 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)	7.335	10.833
V_x (kip)	-3.161	-5.280
V_z (kip)	0.257	0.413
M_x (kipft)	1.043	1.671
M_z (kipft)	43.995	74.339

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.161 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.50334 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.995 \text{ kipft}) + ((-3.161 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0056 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.034 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.040924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.043 \text{ kipft}) + ((0.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16608 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7119 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.034 \text{ ft}), (2.7119 \text{ ft})]$$

$$L_{e,req} = 7.034 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.034 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93787$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.335 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.45844 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.45844 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22922$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.50334 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.0056 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0056 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.0056 \text{ kipft/ft})) + (4 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1652 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.0056 \text{ kipft/ft})) + (3 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.0056 \text{ kipft/ft})) + (2 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.27603 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.0056 \text{ kipft/ft})) + ((-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0918 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1652 \text{ ft})}{2}$$

$$p_a = 0.38739 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27603 \text{ kip/ft}^2)}{(0.38739 \text{ kip/ft}^2)}$$

$$Ratio = 0.71255$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0918 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.97053$$

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

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

Considering z-direction:

$H_o = 0.040924 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.16608 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16608 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.040924 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.16608 \text{ kipft/ft})) + (4 \times (0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.345 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16608 \text{ kipft/ft})) + (3 \times (0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.16608 \text{ kipft/ft})) + (2 \times (0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.030124 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.16608 \text{ kipft/ft})) + ((0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.06817 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.345 \text{ ft})}{2}$$

$$p_a = 0.40087 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.030124 \text{ kip/ft}^2)}{(0.40087 \text{ kip/ft}^2)}$$

$$Ratio = 0.075146$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

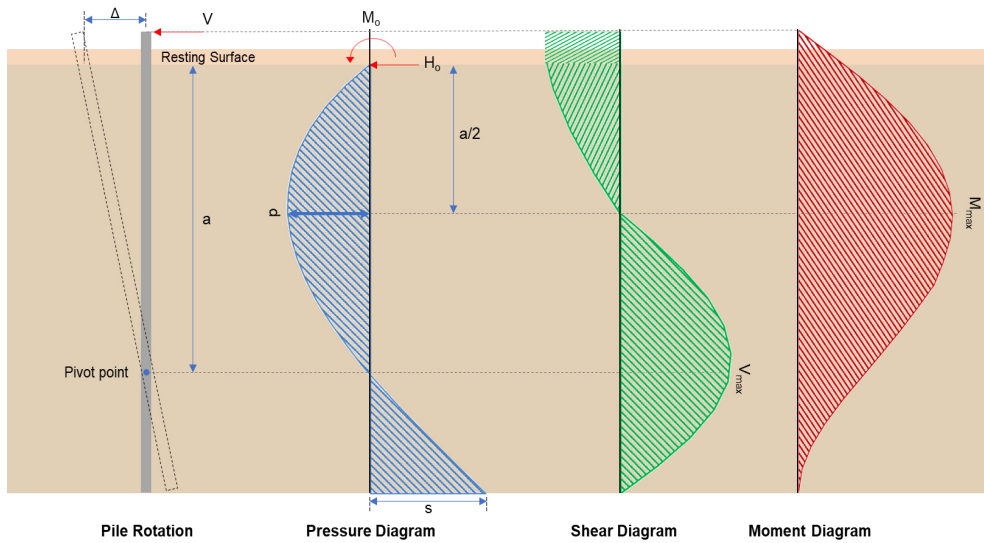
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.06817 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.060595$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.28 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.84076 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(74.339 \text{ kipft}) + ((-5.28 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.837 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.837 \text{ kipft/ft})}{(-0.84076 \text{ kip/ft})}$$

$$E = 14.079 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.837 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.84076 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times 11.837) + (4 \times (-0.84076) \times 7.5)}$$

$$a = \frac{(-0.84076 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.837 \text{ kipft/ft})) + (4 \times (-0.84076 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1638 \text{ ft}$$

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

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

$$V_{max} = ((-0.84076 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.079 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1638 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.079 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1638 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.391 \text{ kip}$$

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

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

$$M_{max} = ((-0.84076 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(14.079 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1638 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.079 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1638 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (14.079 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1638 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 47.922 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.413 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.065764 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.671 \text{ kipft}) + ((0.413 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.26608 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.26608 \text{ kipft/ft})}{(0.065764 \text{ kip/ft})}$$

$$E = 4.046 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.26608 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.065764 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.26608 \text{ kipft/ft})) + (4 \times (0.065764 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3455 \text{ ft}$$

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

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

$$V_{max} = ((0.065764 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.046 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3455 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.046 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3455 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.42618 \text{ kip}$$

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

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

$$M_{max} = ((0.065764 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(4.046 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3455 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.046 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3455 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.046 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3455 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.422 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.833 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.236 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.236 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.833 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0040494$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.833 \text{ kip} \rightarrow 10833 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10833 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.93 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.93 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.93 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

V_{max} = 13.391 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.391 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.1206$$

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

Considering z-direction:

$V_{max} = 0.42618 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.42618 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.0038382$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 47.922 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(47.922 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.192$$

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

Considering z-direction:

$M_{max} = 1.422 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.422 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0056972$$

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

REFERENCES	CALCULATIONS	RESULTS
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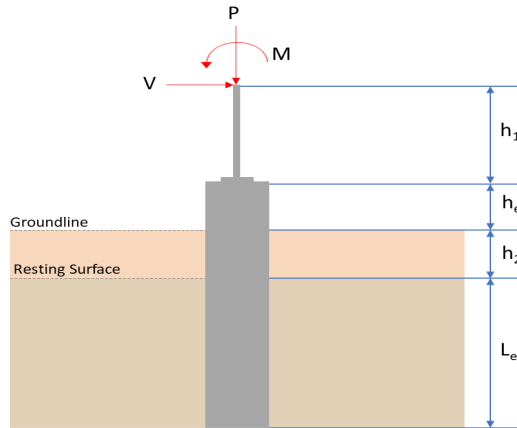
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
 $b = 48$ in - Pile width
 $D = 48$ in - Pile depth
 $L = 7.5$ 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)	7.335	10.833
V_x (kip)	-3.161	-5.279
V_z (kip)	-0.257	-0.413
M_x (kipft)	-1.043	-1.672
M_z (kipft)	43.995	74.339

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.161 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.50334 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.995 \text{ kipft}) + ((-3.161 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0056 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.034 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.040924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.043 \text{ kipft}) + ((-0.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16608 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.0257 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.034 \text{ ft}), (2.0257 \text{ ft})]$$

$$L_{e,req} = 7.034 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.034 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93787$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.335 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.45844 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.45844 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22922$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.50334 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.0056 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0056 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.0056 \text{ kipft/ft})) + (4 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1652 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.0056 \text{ kipft/ft})) + (3 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.0056 \text{ kipft/ft})) + (2 \times (-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.27603 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.0056 \text{ kipft/ft})) + ((-0.50334 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0918 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1652 \text{ ft})}{2}$$

$$p_a = 0.38739 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27603 \text{ kip/ft}^2)}{(0.38739 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.71255$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0918 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97053$$

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

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

Considering z-direction:

$H_o = -0.040924 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.16608 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16608 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.040924 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.16608 \text{ kipft/ft})) + (4 \times (-0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.345 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16608 \text{ kipft/ft})) + (3 \times (-0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.16608 \text{ kipft/ft})) + (2 \times (-0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.0075851 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.16608 \text{ kipft/ft})) + ((-0.040924 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.0026921 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.345 \text{ ft})}{2}$$

$$p_a = 0.40087 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0075851 \text{ kip/ft}^2)}{(0.40087 \text{ kip/ft}^2)}$$

$$Ratio = -0.018922$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

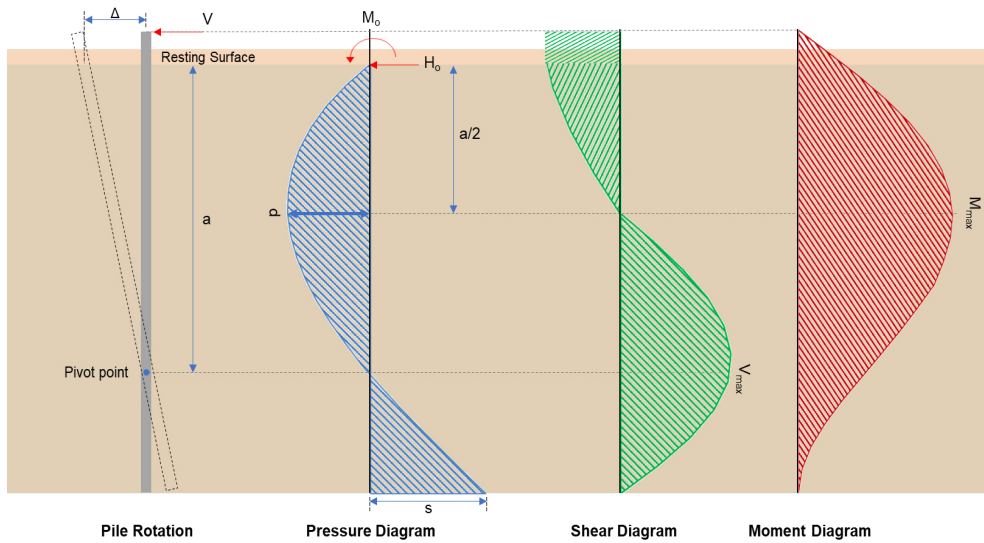
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0026921 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.002393$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.279 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.84061 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(74.339 \text{ kipft}) + ((-5.279 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.837 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.837 \text{ kipft/ft})}{(-0.84061 \text{ kip/ft})}$$

$$E = 14.082 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.837 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.84061 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.837 \text{ kipft/ft})) + (4 \times (-0.84061 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(6 \times (11.837 \text{ kipft/ft})) + (4 \times (-0.84061 \text{ kip/ft}) \times (7.5 \text{ ft}))}{(6 \times (11.837 \text{ kipft/ft})) + (4 \times (-0.84061 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1638 \text{ ft}$$

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

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

$$V_{max} = ((-0.84061 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.082 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1638 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.082 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1638 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.39 \text{ kip}$$

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

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

$$M_{max} = ((-0.84061 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(14.082 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1638 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.082 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1638 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (14.082 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1638 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 47.921 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.413 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.065764 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.672 \text{ kipft}) + ((-0.413 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.26624 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.26624 \text{ kipft/ft})}{(-0.065764 \text{ kip/ft})}$$

$$E = 4.0484 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.26624 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.065764 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.26624 \text{ kipft/ft})) + (4 \times (-0.065764 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3454 \text{ ft}$$

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

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

$$V_{max} = ((-0.065764 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0484 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3454 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.0484 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3454 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.42633 \text{ kip}$$

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

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

$$M_{max} = ((-0.065764 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(4.0484 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3454 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.0484 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3454 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.0484 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3454 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.4226 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.833 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.236 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.236 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.833 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0040494$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.833 \text{ kip} \rightarrow 10833 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10833 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.93 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.93 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.93 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

V_{max} = 13.39 kip - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(13.39 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.12059$$

Considering z-direction:

$V_{max} = 0.42633 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.42633 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.0038396$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 47.921 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(47.921 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.192$$

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

Considering z-direction:

$M_{max} = 1.4226 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.4226 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0056994$$

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

REFERENCES	CALCULATIONS	RESULTS
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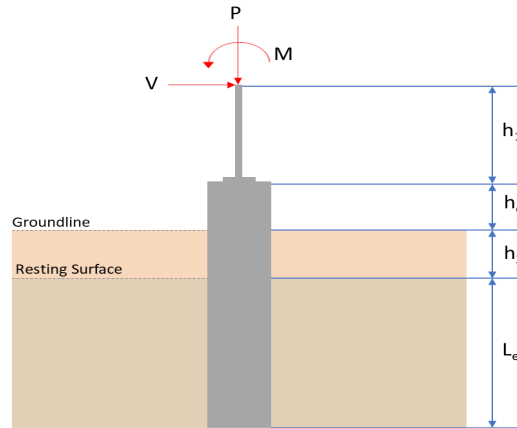
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.940	13.227
V_x (kip)	-3.756	-6.262
V_z (kip)	-0.017	-0.027
M_x (kipft)	-0.071	-0.114
M_z (kipft)	52.508	89.254

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.756 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.508 \text{ kipft}) + ((-3.756 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3611 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3906 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.002707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.071 \text{ kipft}) + ((-0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011306 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.91123 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.3906 \text{ ft}), (0.91123 \text{ ft})]$$

$$L_{e,req} = 7.391 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(7.391 \text{ ft})}{(8 \text{ ft})}$$

$$Ratio = 0.92388$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.94 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55875 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55875 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27937$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59809 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.3611 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3611 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.3611 \text{ kipft/ft})) + (4 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5174 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.3611 \text{ kipft/ft})) + (3 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.3611 \text{ kipft/ft})) + (2 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.27529 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.3611 \text{ kipft/ft})) + ((-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1191 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5174 \text{ ft})}{2}$$

$$p_a = 0.41381 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27529 \text{ kip/ft}^2)}{(0.41381 \text{ kip/ft}^2)}$$

$$Ratio = 0.66526$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1191 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.93262$$

Status: **PASS**
Ratio: **0.670**

Status: **PASS**
Ratio: **0.930**

Considering z-direction:

$H_o = -0.002707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011306 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.011306 \text{ kipft/ft})) + (4 \times (-0.002707 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7072 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.011306 \text{ kipft/ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 [(3 \times (0.011306 \text{ kipft/ft})) + (2 \times (-0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.00048631 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.011306 \text{ kipft/ft})) + ((-0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.00008957 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7072 \text{ ft})}{2}$$

$$p_a = 0.42804 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00048631 \text{ kip/ft}^2)}{(0.42804 \text{ kip/ft}^2)}$$

$$Ratio = -0.0011361$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

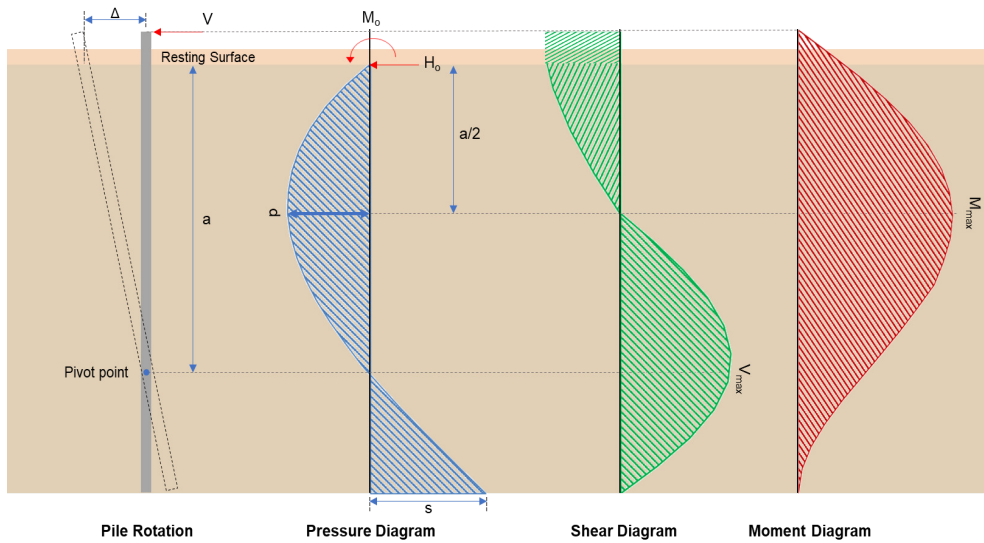
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00008957 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.000074642$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.262 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.99713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(89.254 \text{ kipft}) + ((-6.262 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.212 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.212 \text{ kipft/ft})}{(-0.99713 \text{ kip/ft})}$$

$$E = 14.253 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.212 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.212 \text{ kipft/ft})) + (4 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(-0.99713 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (14.212 \text{ kipft/ft})) + (4 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5149 \text{ ft}$$

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

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

$$V_{max} = ((-0.99713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5149 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5149 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.206 \text{ kip}$$

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

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

$$M_{max} = ((-0.99713 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(14.253 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 57.924 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.027 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0042994 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.114 \text{ kipft}) + ((-0.027 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018153 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018153 \text{ kipft/ft})}{(-0.0042994 \text{ kip/ft})}$$

$$E = 4.2222 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018153 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0042994 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.018153 \text{ kipft/ft})) + (4 \times (-0.0042994 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7054 \text{ ft}$$

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

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

$$V_{max} = ((-0.0042994 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7054 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7054 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.02751 \text{ kip}$$

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

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

$$M_{max} = ((-0.0042994 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2222 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.097758 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.227 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.157 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.157 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.227 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0049443$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.227 \text{ kip} \rightarrow 13227 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13227 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.25 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.25 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.25 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.25 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.24 \text{ kip}$$

Considering x-direction:

V_{max} = 15.206 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.206 \text{ kip})}{(111.24 \text{ kip})}$$

$$Ratio = 0.13669$$

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

Considering z-direction:

$V_{max} = 0.02751 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.02751 \text{ kip})}{(111.24 \text{ kip})}$$

$$Ratio = 0.00024729$$

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:

$M_{max} = 57.924 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(57.924 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.23207$$

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

Considering z-direction:

$M_{max} = 0.097758 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.097758 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00039166$$

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

REFERENCES	CALCULATIONS	RESULTS
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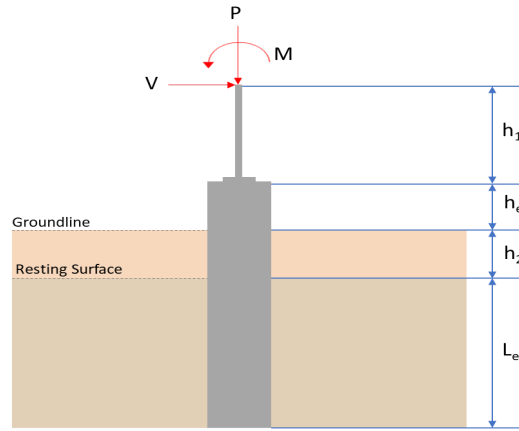
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.940	13.227
V_x (kip)	-3.756	-6.262
V_z (kip)	0.017	0.027
M_x (kipft)	0.071	0.114
M_z (kipft)	52.508	89.254

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.756 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.508 \text{ kipft}) + ((-3.756 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3611 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3906 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.002707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.071 \text{ kipft}) + ((0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011306 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0232 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3906 \text{ ft}), (1.0232 \text{ ft})]$$

$$L_{e,req} = 7.391 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.391 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.92388$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.94 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55875 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55875 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27937$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59809 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.3611 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3611 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.3611 \text{ kipft/ft})) + (4 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5174 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.3611 \text{ kipft/ft})) + (3 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.3611 \text{ kipft/ft})) + (2 \times (-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.27529 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.3611 \text{ kipft/ft})) + ((-0.59809 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1191 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5174 \text{ ft})}{2}$$

$$p_a = 0.41381 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27529 \text{ kip/ft}^2)}{(0.41381 \text{ kip/ft}^2)}$$

$$Ratio = 0.66526$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1191 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.93262$$

Status: **PASS**
Ratio: **0.670**

Status: **PASS**
Ratio: **0.930**

Considering z-direction:

$H_o = 0.002707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011306 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.011306 \text{ kipft/ft})) + (4 \times (0.002707 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7072 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.011306 \text{ kipft/ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.011306 \text{ kipft/ft})) + (2 \times (0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.0018424 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.011306 \text{ kipft/ft})) + ((0.002707 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.0041501 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7072 \text{ ft})}{2}$$

$$p_a = 0.42804 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0018424 \text{ kip/ft}^2)}{(0.42804 \text{ kip/ft}^2)}$$

$$Ratio = 0.0043043$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

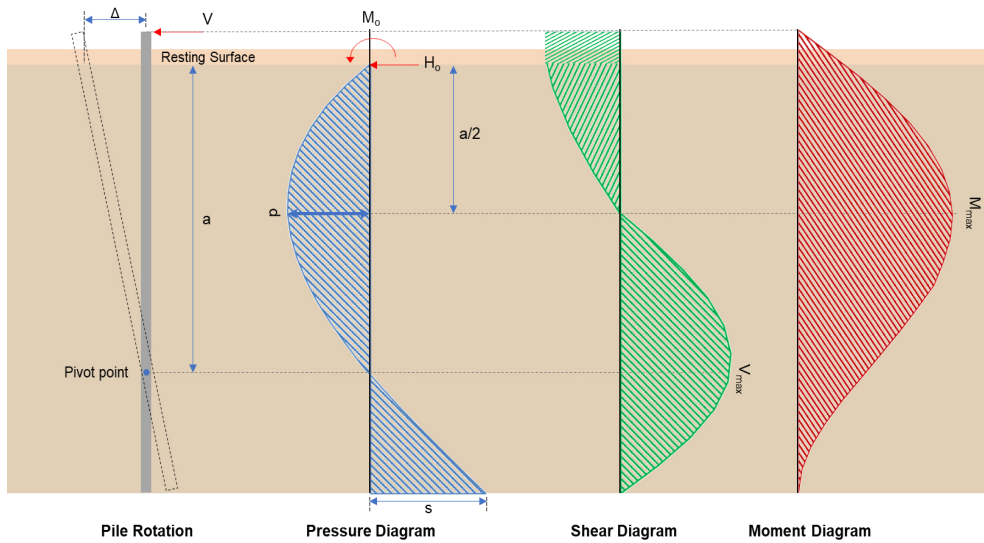
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0041501 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.0034584$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.262 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.99713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(89.254 \text{ kipft}) + ((-6.262 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.212 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.212 \text{ kipft/ft})}{(-0.99713 \text{ kip/ft})}$$

$$E = 14.253 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.212 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.212 \text{ kipft/ft})) + (4 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(-0.99713 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (14.212 \text{ kipft/ft})) + (4 \times (-0.99713 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5149 \text{ ft}$$

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

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

$$V_{max} = ((-0.99713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5149 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5149 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.206 \text{ kip}$$

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

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

$$M_{max} = ((-0.99713 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(14.253 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.253 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5149 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 57.924 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.027 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0042994 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.114 \text{ kipft}) + ((0.027 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018153 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018153 \text{ kipft/ft})}{(0.0042994 \text{ kip/ft})}$$

$$E = 4.2222 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018153 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0042994 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.018153 \text{ kipft/ft})) + (4 \times (0.0042994 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7054 \text{ ft}$$

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

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

$$V_{max} = ((0.0042994 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7054 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7054 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.02751 \text{ kip}$$

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

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

$$M_{max} = ((0.0042994 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2222 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2222 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7054 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.097758 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{13.227 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.157 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.157 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Min[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Min[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.227 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0049443$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.227 \text{ kip} \rightarrow 13227 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13227 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.25 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.25 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.25 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.25 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.24 \text{ kip}$$

Considering x-direction:

V_{max} = 15.206 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.206 \text{ kip})}{(111.24 \text{ kip})}$$

$$Ratio = 0.13669$$

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

Considering z-direction:

$V_{max} = 0.02751 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.02751 \text{ kip})}{(111.24 \text{ kip})}$$

$$Ratio = 0.00024729$$

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:

$M_{max} = 57.924 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(57.924 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.23207$$

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

Considering z-direction:

$M_{max} = 0.097758 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.097758 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00039166$$

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