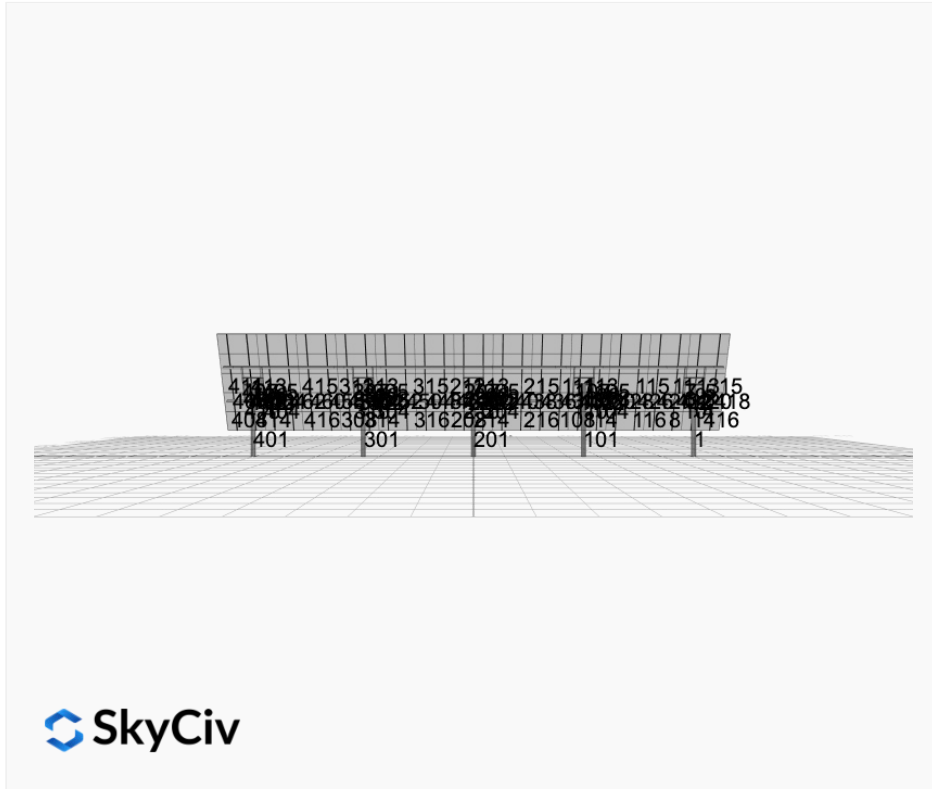


Project Details



Project Name: Giesey Opt3 - V1Jb **Date:** Wed Jul 16 2025
Location: 8385 Jordan Rd, Melvin, MI 48454, USA **Number of Modules:** 65
Unique ID: 5P-22.5-10TOP-HD-24-L-5Hx13W-AGB1 **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	21.58 ft
Array Dimensions E/W	102.81 ft
Winter Tilt Angle	65
Front Edge Clearance	5 ft

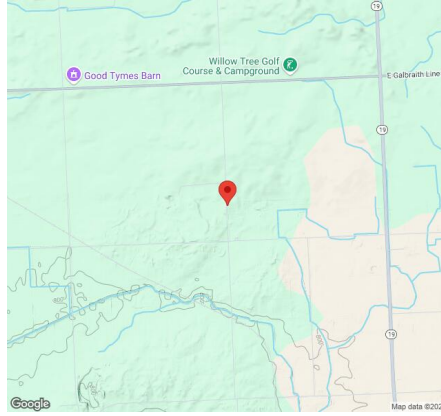
MT Solar Bill of Materials (5P-22.5-10TOP-HD-24-L-5Hx13W-AGB1)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	16
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (259in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: 8385 Jordan Rd, Melvin, MI 48454, USA

Array Specification

Duty Classification:	HD
Module Width:	51.30 in
Module Length:	93.90in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	65
Front Edge Clearance:	5
Total Array Height at Tilt:	24.56 ft
Total Frame Length:	101.50 ft
Module Info/Notes:	Hanersun HN21H-66HT
Array Dimensions N/S:	21.58 ft
Array Dimensions E/W:	102.81 ft
Rail Length:	259.00 in
Rail Spacing:	3.95 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	14.78 ft
Number of Poles:	5
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.25 ft Pile 2: 9.00 ft Pile 3: 9.00 ft Pile 4: 9.00 ft Pile 5: 8.25 ft
Foundation Volume:	25.778 y ³

Site Info

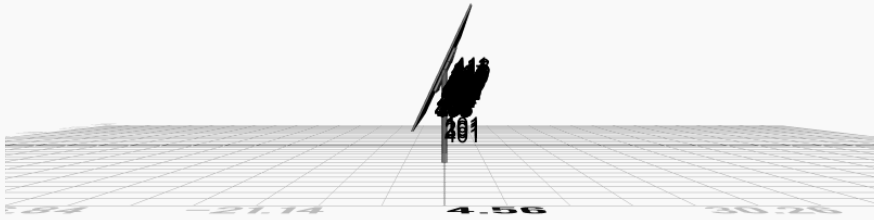
Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	8385 Jordan Rd, Melvin, MI 48454, USA
Wind Speed:	101 mph

Snow Load:

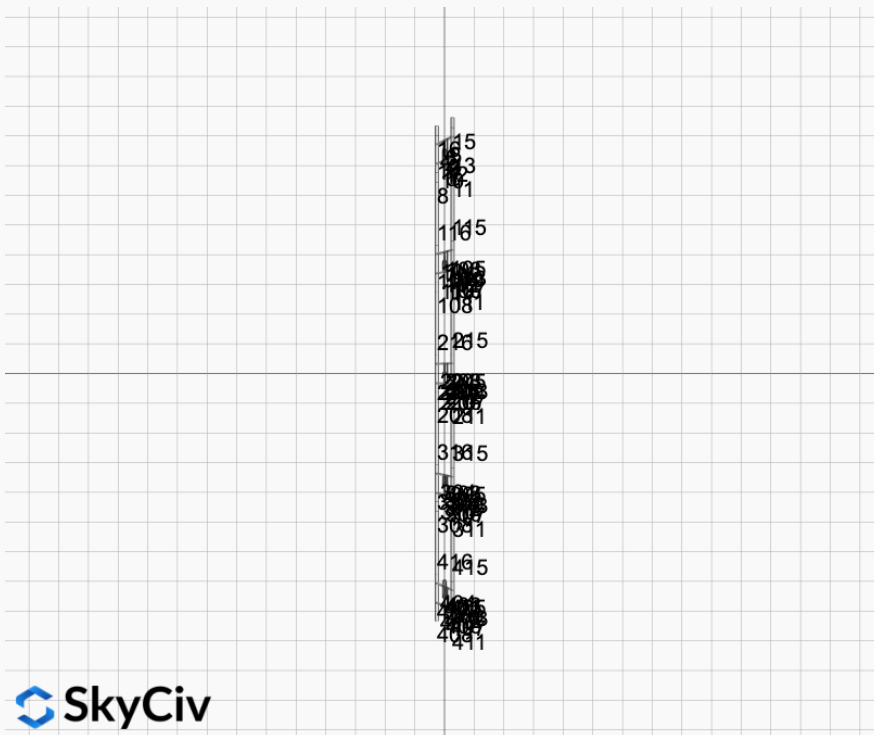
25 psf

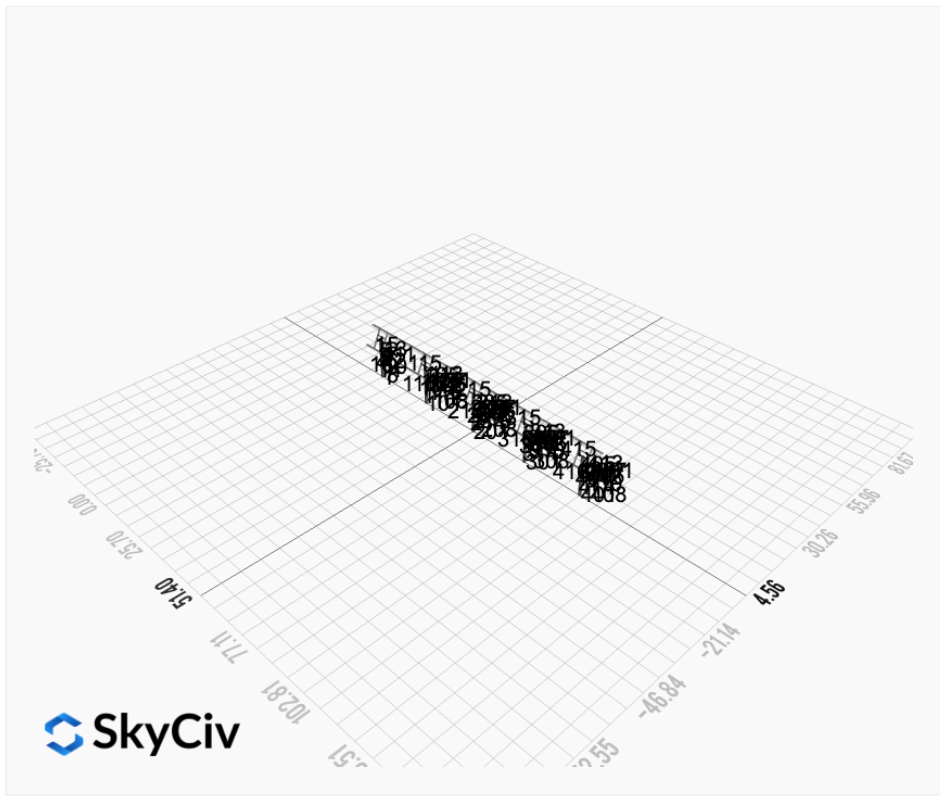
Design Disclaimer

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.



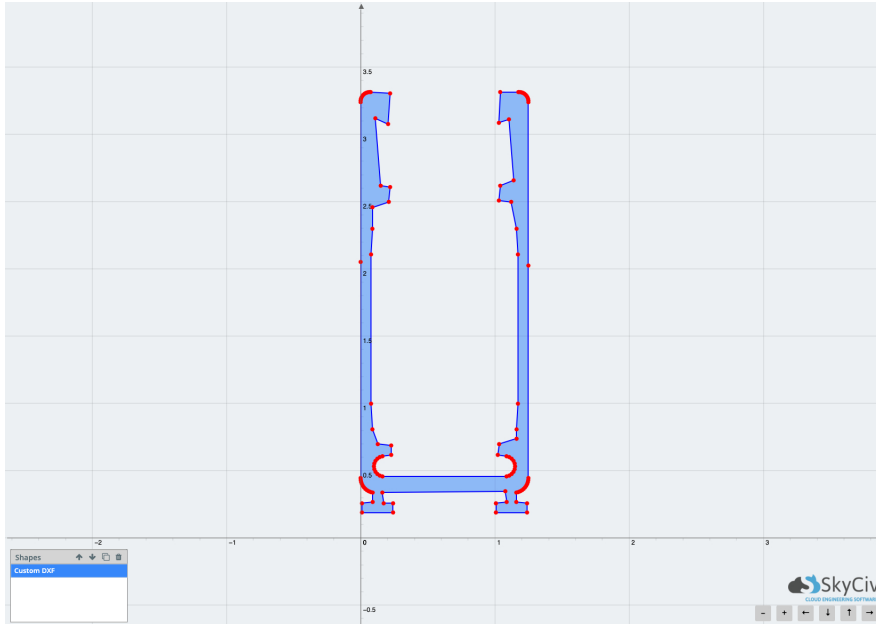
 SkyCiv





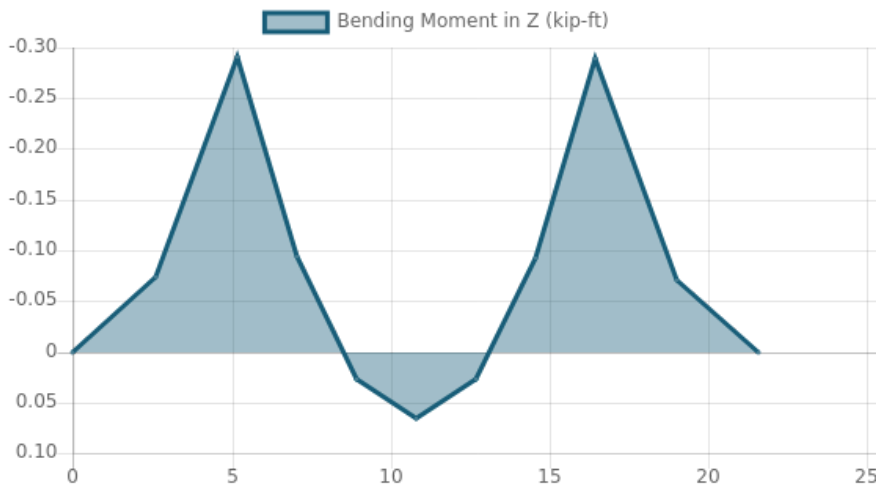
Rail Design Check

Rail Length: 21.58333333333332 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.95416666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0023 kip/ft
Snow (Y): -0.0049 kip/ft
Wind uplift Case A: 0.0744 kip/ft
Wind downforce Case A: 0.0744 kip/ft
Dead (Panel load) (X): 0.0073 kip/ft
Dead (Panel load) (Y): -0.0156 kip/ft

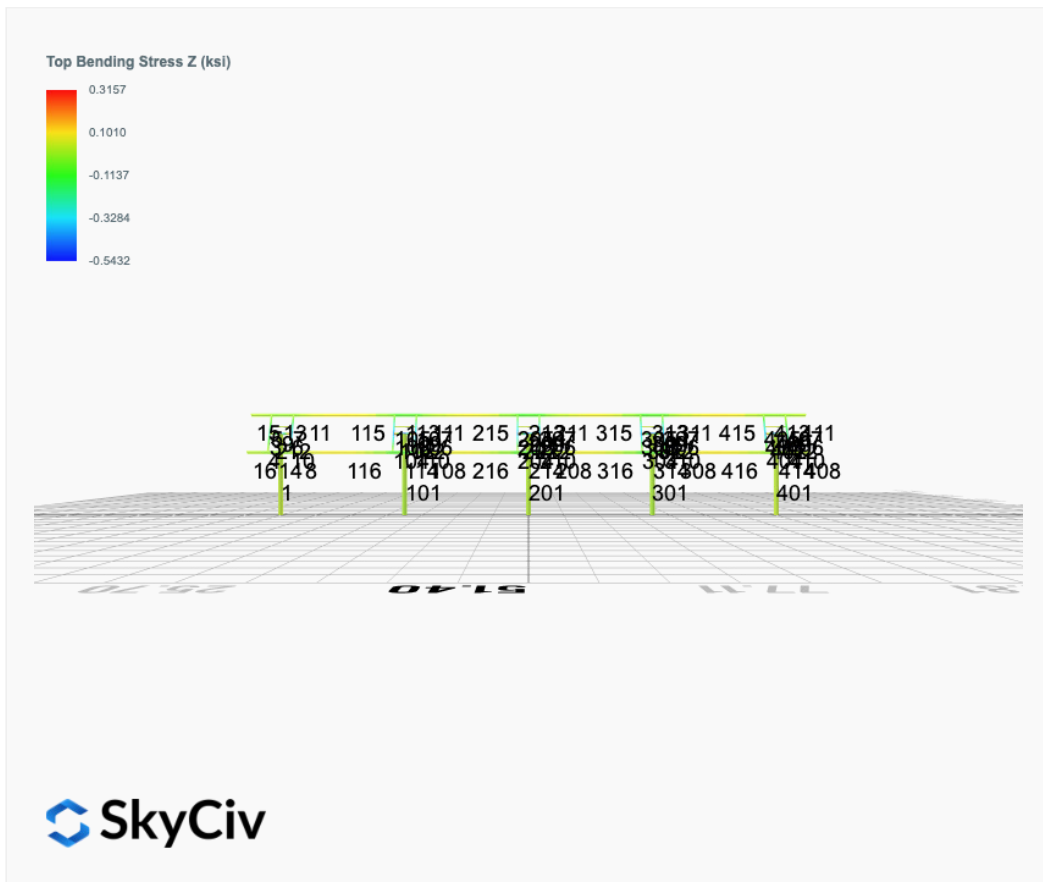
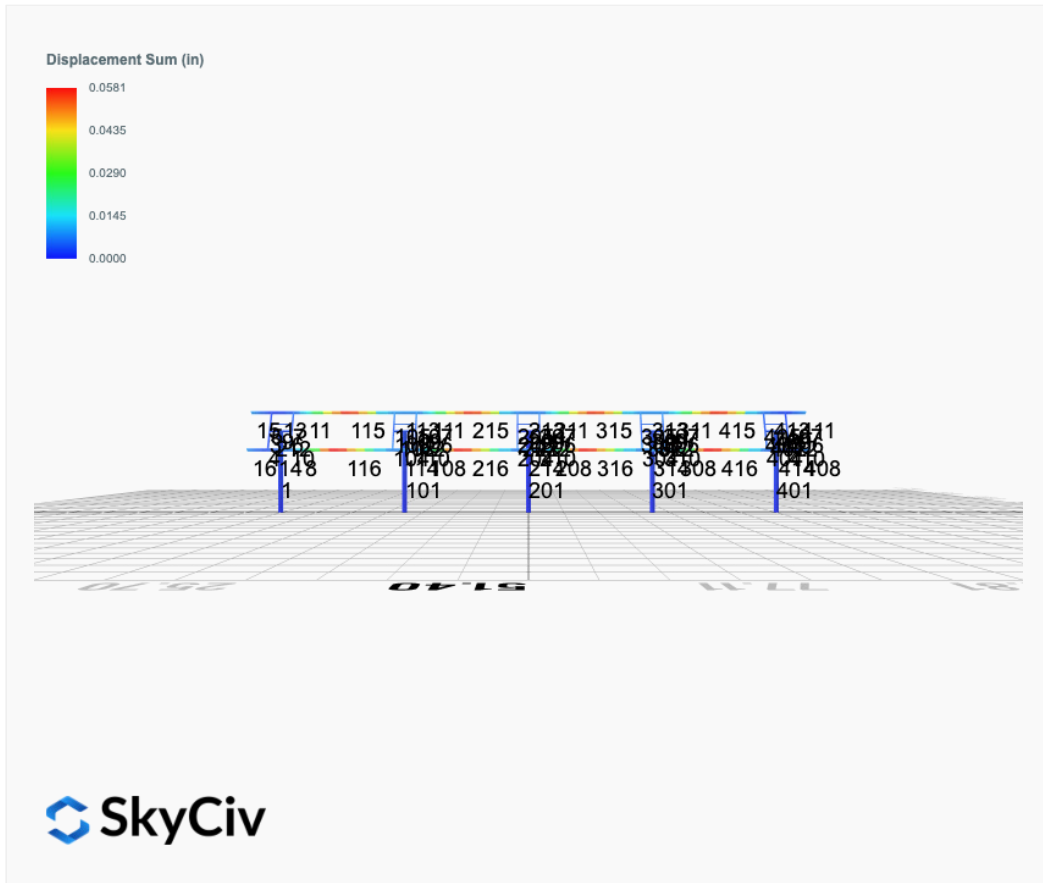


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	27.53330303	0.798	PASS
Material Yield	34.5	27.53330303	0.798	PASS
Material Strength	37	27.53330303	0.744	PASS

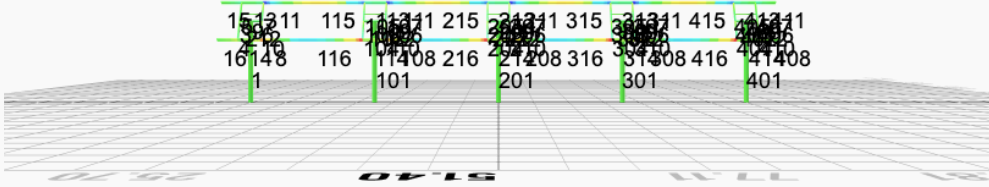
Member 1, ULS: 1. 1.4D



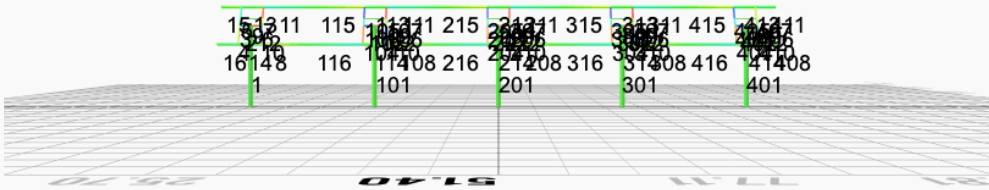
FEM Results (Envelope Worst Case for each member)



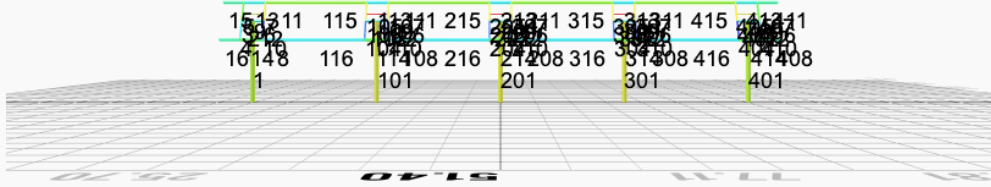
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 2. D + L	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 3. D + (S or Lr or R)	0.0350	2.9655	0.1110	0.4961	-0.2045	-0.4888
ULS: 3. D + (S or Lr or R)	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0342	2.9127	0.1083	0.4839	-0.1995	-0.4764
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 5b. D + 0.7E	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0342	2.9127	0.1083	0.4839	-0.1995	-0.4764
ULS: 8. 0.6D + 0.7E	0.0189	1.6525	0.0601	0.2685	-0.1107	-0.2636
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7006	4.4685	0.3165	1.3292	-3.2209	55.3417
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7606	1.0413	-0.1134	-0.4218	2.8147	-55.4455
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7649	4.1985	0.2706	1.1452	-2.4769	41.3594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0342	2.9127	0.1083	0.4839	-0.1995	-0.4764
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8309	1.6280	-0.0519	-0.1681	2.0499	-41.7310
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0342	2.9127	0.1083	0.4839	-0.1995	-0.4764
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7675	4.0399	0.2624	1.1088	-2.4618	41.3964
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8283	1.4695	-0.0600	-0.2045	2.0650	-41.6940
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0316	2.7542	0.1001	0.4476	-0.1844	-0.4394
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7132	3.3668	0.2765	1.1502	-3.1471	55.5175
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0189	1.6525	0.0601	0.2685	-0.1107	-0.2636
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7479	-0.0604	-0.1535	-0.6008	2.8885	-55.2698
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0189	1.6525	0.0601	0.2685	-0.1107	-0.2636

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	6.2686
Shear X	-6.2568
Shear Z	0.4884
Moment X	2.0398
Moment Y (Twist)	5.3173
Moment Z	93.2271

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	4.4685
Shear X	-3.7606
Shear Z	0.3165
Moment X	1.3292
Moment Y (Twist)	3.2209
Moment Z	55.5175

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	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 2. D + L	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 3. D + (S or Lr or R)	-0.0335	3.7041	-0.0021	-0.0127	0.0553	0.4865
ULS: 3. D + (S or Lr or R)	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0327	3.6331	-0.0020	-0.0124	0.0539	0.4749

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 5b. D + 0.7E	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0327	3.6331	-0.0020	-0.0124	0.0539	0.4749
ULS: 8. 0.6D + 0.7E	-0.0181	2.0522	-0.0011	-0.0069	0.0299	0.2642
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.1043	5.8111	0.0155	0.0509	-0.2974	75.7802
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 5a. D + 0.6W_Wind uplift Case A only	5.0478	1.0275	-0.0175	-0.0672	0.3728	-73.6347
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8383	5.4262	0.0110	0.0344	-0.2065	56.9798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0327	3.6331	-0.0020	-0.0124	0.0539	0.4749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.7758	1.8386	-0.0138	-0.0542	0.2961	-55.0814
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0327	3.6331	-0.0020	-0.0124	0.0539	0.4749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8358	5.2134	0.0111	0.0353	-0.2106	56.9452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.7783	1.6257	-0.0136	-0.0532	0.2921	-55.1159
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0302	3.4203	-0.0019	-0.0114	0.0499	0.4404
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.0922	4.4430	0.0162	0.0554	-0.3173	75.6040
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0181	2.0522	-0.0011	-0.0069	0.0299	0.2642
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	5.0599	-0.3406	-0.0168	-0.0626	0.3529	-73.8109
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0181	2.0522	-0.0011	-0.0069	0.0299	0.2642

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.2296
Shear X	-8.4912
Shear Z	-0.0298
Moment X	-0.1118
Moment Y (Twist)	0.6208
Moment Z	127.3775

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.8111
Shear X	-5.1043
Shear Z	-0.0175
Moment X	-0.0672
Moment Y (Twist)	0.3728
Moment Z	75.7802

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	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 2. D + L	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 3. D + (S or Lr or R)	-0.0030	3.6874	0.0000	-0.0001	0.0001	0.0869
ULS: 3. D + (S or Lr or R)	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0029	3.6168	0.0000	-0.0001	0.0001	0.0852
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 5b. D + 0.7E	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0029	3.6168	0.0000	-0.0001	0.0001	0.0852
ULS: 8. 0.6D + 0.7E	-0.0016	2.0431	0.0000	-0.0000	0.0000	0.0480
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.0987	5.7840	0.0000	-0.0001	0.0000	76.2342
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 5a. D + 0.6W_Wind uplift Case A only	5.0918	1.0273	0.0000	-0.0001	0.0000	-74.7029
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8249	5.4009	0.0000	-0.0001	0.0000	57.2008
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0029	3.6168	0.0000	-0.0001	0.0001	0.0852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.8179	1.8334	0.0000	-0.0001	0.0000	-56.0020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0029	3.6168	0.0000	-0.0001	0.0001	0.0852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8247	5.1893	0.0000	-0.0001	0.0000	57.1957
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.8182	1.6218	0.0000	-0.0001	0.0000	-56.0072
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0027	3.4052	0.0000	-0.0001	0.0000	0.0800
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.0976	4.4219	0.0000	-0.0000	0.0000	76.2022
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0016	2.0431	0.0000	-0.0000	0.0000	0.0480
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	5.0928	-0.3347	0.0000	-0.0000	0.0000	-74.7349
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0016	2.0431	0.0000	-0.0000	0.0000	0.0480

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.1932
Shear X	-8.4989
Shear Z	0.0000
Moment X	-0.0008
Moment Y (Twist)	0.0004
Moment Z	128.3815

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.7840
Shear X	-5.0987
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	76.2342

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	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 2. D + L	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 3. D + (S or Lr or R)	-0.0335	3.7041	0.0021	0.0126	-0.0552	0.4864
ULS: 3. D + (S or Lr or R)	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0327	3.6331	0.0020	0.0123	-0.0538	0.4749
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 5b. D + 0.7E	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0327	3.6331	0.0020	0.0123	-0.0538	0.4749
ULS: 8. 0.6D + 0.7E	-0.0181	2.0522	0.0011	0.0068	-0.0299	0.2642
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.1043	5.8111	-0.0155	-0.0510	0.2975	75.7802
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 5a. D + 0.6W_Wind uplift Case A only	5.0478	1.0275	0.0176	0.0671	-0.3727	-73.6347
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8383	5.4262	-0.0110	-0.0345	0.2066	56.9798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0327	3.6331	0.0020	0.0123	-0.0538	0.4749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.7758	1.8386	0.0138	0.0541	-0.2961	-55.0814
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0327	3.6331	0.0020	0.0123	-0.0538	0.4749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8358	5.2134	-0.0111	-0.0354	0.2107	56.9452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.7783	1.6257	0.0136	0.0531	-0.2920	-55.1159
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0302	3.4203	0.0019	0.0113	-0.0498	0.4404

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.0922	4.4430	-0.0162	-0.0555	0.3174	75.6040
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0181	2.0522	0.0011	0.0068	-0.0299	0.2642
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	5.0599	-0.3406	0.0168	0.0625	-0.3528	-73.8109
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0181	2.0522	0.0011	0.0068	-0.0299	0.2642

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	8.2296
Shear X	-8.4912
Shear Z	0.0298
Moment X	0.1128
Moment Y (Twist)	0.6209
Moment Z	127.3777

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	5.8111
Shear X	-5.1043
Shear Z	0.0176
Moment X	0.0671
Moment Y (Twist)	0.3727
Moment Z	75.7802

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 2. D + L	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 3. D + (S or Lr or R)	0.0350	2.9655	-0.1110	-0.4964	0.2046	-0.4887
ULS: 3. D + (S or Lr or R)	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0342	2.9127	-0.1083	-0.4843	0.1996	-0.4763
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 5b. D + 0.7E	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0342	2.9127	-0.1083	-0.4843	0.1996	-0.4763
ULS: 8. 0.6D + 0.7E	0.0189	1.6525	-0.0601	-0.2687	0.1107	-0.2636
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7006	4.4685	-0.3165	-1.3296	3.2210	55.3418
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7606	1.0413	0.1134	0.4215	-2.8147	-55.4455
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7649	4.1984	-0.2706	-1.1455	2.4769	41.3595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0342	2.9127	-0.1083	-0.4843	0.1996	-0.4763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8309	1.6280	0.0519	0.1677	-2.0498	-41.7309
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0342	2.9127	-0.1083	-0.4843	0.1996	-0.4763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7675	4.0399	-0.2624	-1.1092	2.4619	41.3965
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8283	1.4695	0.0600	0.2041	-2.0649	-41.6939
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0316	2.7541	-0.1001	-0.4479	0.1845	-0.4393
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7132	3.3668	-0.2765	-1.1504	3.1472	55.5175
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0189	1.6525	-0.0601	-0.2687	0.1107	-0.2636
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7479	-0.0604	0.1535	0.6006	-2.8885	-55.2697
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0189	1.6525	-0.0601	-0.2687	0.1107	-0.2636

Worst Case Reactions LRFD

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

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

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	6.2685
Shear X	-6.2568
Shear Z	-0.4884
Moment X	-2.0416
Moment Y (Twist)	5.3178
Moment Z	93.2285

Result	Value (kip, kip-ft)
Axial	4.4685
Shear X	-3.7606
Shear Z	-0.3165
Moment X	-1.3296
Moment Y (Twist)	3.2210
Moment Z	55.5175

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Giesey Opt3 - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
11	10in Pipe Sch 40	10.75	0.36					

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

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

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

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	23.83	6.12	40.24	43.62
114	133.20	85.85	23.73	6.12	40.24	43.62
115	133.20	46.28	11.91	6.12	40.24	43.62
116	133.20	46.28	12.39	6.12	40.24	43.62
201	535.87	252.74	147.68	147.68	160.76	160.76
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	23.49	6.12	40.24	43.62
214	133.20	85.85	23.50	6.12	40.24	43.62
215	133.20	46.28	12.64	6.12	40.24	43.62
216	133.20	46.28	12.70	6.12	40.24	43.62
301	535.87	252.74	147.68	147.68	160.76	160.76
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.83	6.12	40.24	43.62
314	133.20	85.85	23.73	6.12	40.24	43.62
315	133.20	46.28	12.24	6.12	40.24	43.62
316	133.20	46.28	12.52	6.12	40.24	43.62
401	535.87	252.74	147.68	147.68	160.76	160.76
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28

407	116.10	114.23	15.79	11.10	42.08	23.28
408	133.20	102.39	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	102.39	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	24.96	6.12	40.24	43.62
414	133.20	85.85	24.70	6.12	40.24	43.62
415	133.20	46.28	11.93	6.12	40.24	43.62
416	133.20	46.28	11.91	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.025	0.631	0.035	0.039	0.003	0.658	#13	0.507	Not Required	Pass
2	0.002	0.076	0.138	0.027	0.036	0.216	#13	0.035	Not Required	Pass
3	0.004	0.316	0.027	0.029	0.007	0.322	#13	0.045	Not Required	Pass
4	0.004	0.320	0.068	0.032	0.017	0.389	#13	0.080	Not Required	Pass
5	0.003	0.197	0.040	0.032	0.011	0.207	#13	0.074	Not Required	Pass
6	0.008	0.642	0.067	0.067	0.017	0.709	#13	0.045	Not Required	Pass
7	0.008	0.398	0.142	0.064	0.034	0.425	#13	0.074	Not Required	Pass
8	0.006	0.114	0.161	0.040	0.013	0.184	#15	0.095	Not Required	Pass
9	0.007	0.069	0.103	0.006	0.006	0.138	#13	0.204	Not Required	Pass
10	0.009	0.605	0.153	0.060	0.034	0.640	#13	0.080	Not Required	Pass
11	0.004	0.098	0.163	0.043	0.013	0.167	#23	0.095	Not Required	Pass
12	0.002	0.312	0.381	0.067	0.072	0.693	#13	0.053	Not Required	Pass
13	0.006	0.119	0.338	0.053	0.015	0.418	#15	0.286	Not Required	Pass
14	0.006	0.099	0.335	0.050	0.015	0.392	#15	0.190	Not Required	Pass
15	0.000	0.014	0.024	0.012	0.003	0.036	#13	Not Required	Not Required	Pass
16	0.000	0.014	0.024	0.012	0.003	0.036	#13	Not Required	Not Required	Pass
101	0.033	0.863	0.002	0.053	0.000	0.879	#13	0.507	Not Required	Pass
102	0.003	0.257	0.351	0.063	0.069	0.609	#13	0.035	Not Required	Pass
103	0.008	0.620	0.040	0.061	0.002	0.660	#13	0.045	Not Required	Pass
104	0.007	0.648	0.145	0.065	0.030	0.741	#13	0.080	Not Required	Pass
105	0.007	0.384	0.151	0.061	0.038	0.423	#13	0.074	Not Required	Pass
106	0.008	0.658	0.039	0.066	0.004	0.692	#13	0.045	Not Required	Pass
107	0.007	0.409	0.147	0.065	0.037	0.447	#13	0.074	Not Required	Pass
108	0.006	0.060	0.152	0.044	0.013	0.196	#13	0.095	Not Required	Pass
109	0.014	0.031	0.069	0.001	0.001	0.105	#13	0.204	Not Required	Pass
110	0.007	0.660	0.139	0.066	0.029	0.740	#13	0.080	Not Required	Pass
111	0.005	0.051	0.157	0.043	0.013	0.182	#13	0.095	Not Required	Pass
112	0.003	0.272	0.375	0.063	0.073	0.648	#13	0.035	Not Required	Pass
113	0.007	0.203	0.342	0.055	0.016	0.490	#13	0.286	Not Required	Pass
114	0.009	0.246	0.339	0.058	0.016	0.540	#13	0.286	Not Required	Pass
115	0.012	0.539	0.182	0.045	0.013	0.706	#13	0.601	Not Required	Pass
116	0.006	0.517	0.179	0.048	0.013	0.683	#13	0.601	Not Required	Pass
201	0.032	0.869	0.000	0.053	0.000	0.886	#13	0.507	Not Required	Pass
202	0.003	0.265	0.365	0.063	0.071	0.632	#13	0.035	Not Required	Pass
203	0.008	0.652	0.039	0.065	0.002	0.691	#13	0.045	Not Required	Pass
204	0.007	0.640	0.139	0.064	0.029	0.723	#13	0.080	Not Required	Pass
205	0.007	0.405	0.147	0.065	0.037	0.447	#13	0.074	Not Required	Pass

203	0.007	0.405	0.147	0.065	0.037	0.441	#13	0.074	Not Required	Pass
206	0.008	0.652	0.039	0.065	0.002	0.690	#13	0.045	Not Required	Pass
207	0.007	0.405	0.147	0.065	0.037	0.441	#13	0.074	Not Required	Pass
208	0.006	0.065	0.152	0.044	0.013	0.204	#13	0.095	Not Required	Pass
209	0.014	0.031	0.068	0.001	0.000	0.099	#13	0.204	Not Required	Pass
210	0.007	0.640	0.139	0.064	0.029	0.723	#13	0.080	Not Required	Pass
211	0.005	0.064	0.156	0.045	0.013	0.206	#13	0.095	Not Required	Pass
212	0.003	0.265	0.365	0.063	0.071	0.632	#13	0.035	Not Required	Pass
213	0.007	0.256	0.332	0.056	0.016	0.540	#13	0.286	Not Required	Pass
214	0.009	0.259	0.326	0.055	0.016	0.536	#13	0.286	Not Required	Pass
215	0.012	0.387	0.182	0.045	0.013	0.553	#13	0.601	Not Required	Pass
216	0.006	0.359	0.179	0.044	0.013	0.524	#13	0.601	Not Required	Pass
301	0.033	0.863	0.002	0.053	0.000	0.879	#13	0.507	Not Required	Pass
302	0.003	0.272	0.375	0.063	0.073	0.648	#13	0.035	Not Required	Pass
303	0.008	0.658	0.039	0.066	0.004	0.692	#13	0.045	Not Required	Pass
304	0.007	0.660	0.139	0.066	0.029	0.740	#13	0.080	Not Required	Pass
305	0.007	0.409	0.147	0.065	0.037	0.447	#13	0.074	Not Required	Pass
306	0.008	0.620	0.040	0.061	0.002	0.660	#13	0.045	Not Required	Pass
307	0.007	0.384	0.151	0.061	0.038	0.423	#13	0.074	Not Required	Pass
308	0.006	0.063	0.165	0.048	0.013	0.207	#13	0.095	Not Required	Pass
309	0.014	0.031	0.069	0.001	0.001	0.105	#13	0.204	Not Required	Pass
310	0.007	0.648	0.145	0.065	0.030	0.741	#13	0.080	Not Required	Pass
311	0.004	0.087	0.167	0.045	0.013	0.178	#21	0.095	Not Required	Pass
312	0.003	0.257	0.351	0.063	0.069	0.609	#13	0.035	Not Required	Pass
313	0.007	0.203	0.343	0.055	0.016	0.490	#13	0.286	Not Required	Pass
314	0.009	0.246	0.339	0.058	0.016	0.540	#13	0.286	Not Required	Pass
315	0.012	0.393	0.182	0.043	0.013	0.559	#13	0.601	Not Required	Pass
316	0.006	0.361	0.179	0.044	0.013	0.526	#13	0.601	Not Required	Pass
401	0.025	0.631	0.035	0.039	0.003	0.658	#13	0.507	Not Required	Pass
402	0.002	0.312	0.381	0.067	0.072	0.693	#13	0.053	Not Required	Pass
403	0.008	0.642	0.067	0.067	0.017	0.709	#13	0.045	Not Required	Pass
404	0.009	0.605	0.153	0.060	0.034	0.640	#13	0.080	Not Required	Pass
405	0.008	0.398	0.142	0.064	0.034	0.425	#13	0.074	Not Required	Pass
406	0.004	0.316	0.027	0.029	0.007	0.322	#13	0.045	Not Required	Pass
407	0.003	0.197	0.040	0.032	0.011	0.207	#13	0.074	Not Required	Pass
408	0.000	0.014	0.024	0.012	0.003	0.036	#13	Not Required	Not Required	Pass
409	0.007	0.069	0.103	0.006	0.006	0.138	#13	0.204	Not Required	Pass
410	0.004	0.320	0.068	0.032	0.017	0.389	#13	0.080	Not Required	Pass
411	0.000	0.014	0.024	0.012	0.003	0.036	#13	Not Required	Not Required	Pass
412	0.002	0.076	0.138	0.027	0.036	0.216	#13	0.035	Not Required	Pass
413	0.006	0.119	0.338	0.053	0.015	0.418	#15	0.190	Not Required	Pass
414	0.006	0.099	0.335	0.050	0.015	0.393	#15	0.286	Not Required	Pass
415	0.012	0.543	0.182	0.043	0.013	0.707	#13	0.601	Not Required	Pass
416	0.006	0.540	0.179	0.040	0.013	0.704	#13	0.601	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress

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

REFERENCES	CALCULATIONS	RESULTS
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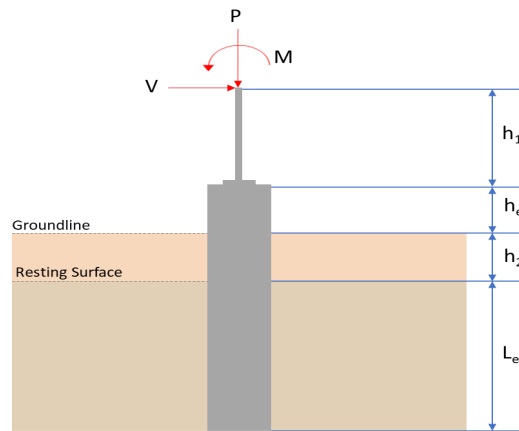
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.469	6.269
V_x (kip)	-3.761	-6.257
V_z (kip)	0.317	0.488
M_x (kipft)	1.329	2.040
M_z (kipft)	55.517	93.227

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,

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

$$H_o = \frac{(-3.761 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59889 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.517 \text{ kipft}) + ((-3.761 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.8403 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5768 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.317 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.050478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.329 \text{ kipft}) + ((0.317 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21162 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.9583 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5768 \text{ ft}), (2.9583 \text{ ft})]$$

$$L_{e,req} = 7.577 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.577 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91842$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.469 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.27931 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.27931 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.13966$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59889 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8403 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8403 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (8.8403 \text{ kipft/ft})) + (4 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

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

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

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

$$p = \frac{0.75 \times [(4 \times (8.8403 \text{ kipft/ft})) + (3 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (8.8403 \text{ kipft/ft})) + (2 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.27936 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.8403 \text{ kipft/ft})) + ((-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1231 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6866 \text{ ft})}{2}$$

$$p_a = 0.4265 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27936 \text{ kip/ft}^2)}{(0.4265 \text{ kip/ft}^2)}$$

$$Ratio = 0.65501$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1231 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.90753$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = 0.050478 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.21162 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21162 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.050478 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.21162 \text{ kipft/ft})) + (4 \times (0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8901 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.21162 \text{ kipft/ft})) + (3 \times (0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 [(3 \times (0.21162 \text{ kipft/ft})) + (2 \times (0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.032977 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.21162 \text{ kipft/ft})) + ((0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.074022 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8901 \text{ ft})}{2}$$

$$p_a = 0.44176 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.032977 \text{ kip/ft}^2)}{(0.44176 \text{ kip/ft}^2)}$$

$$Ratio = 0.074649$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

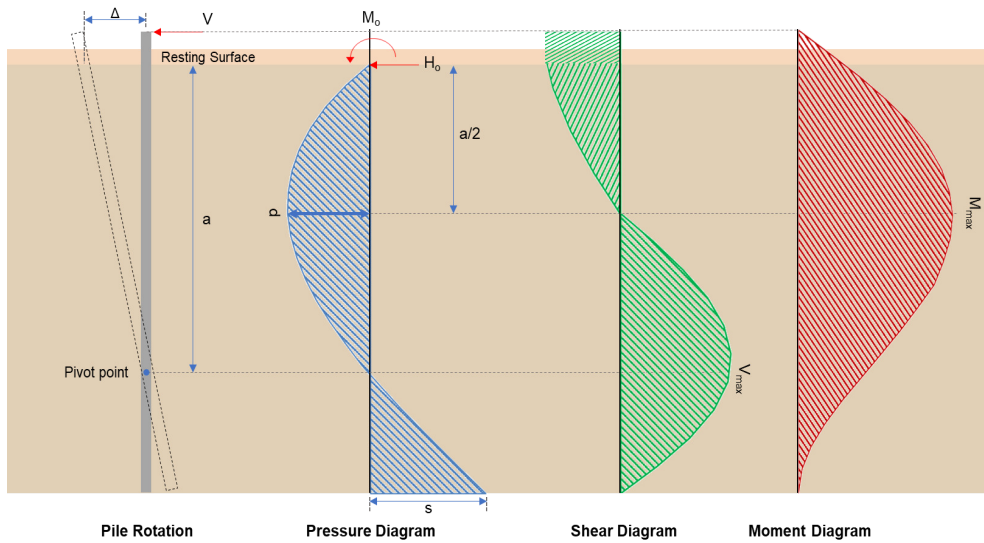
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.074022 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.059816$$

Status: **PASS**
Ratio: **0.070**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.99634 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(93.227 \text{ kipft}) + ((-6.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.845 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.845 \text{ kipft/ft})}{(-0.99634 \text{ kip/ft})}$$

$$E = 14.9 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.845 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (14.845 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = \frac{6 \times (14.845 \text{ kipft/ft}) + (4 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft}))}{\dots}$$

$$a = 5.6854 \text{ ft}$$

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

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

$$V_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6854 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6854 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.365 \text{ kip}$$

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

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

$$M_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(14.9 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 60.395 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.488 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.077707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.04 \text{ kipft}) + ((0.488 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.32484 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.32484 \text{ kipft/ft})}{(0.077707 \text{ kip/ft})}$$

$$E = 4.1803 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.32484 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.077707 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.32484 \text{ kipft/ft})) + (4 \times (0.077707 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8906 \text{ ft}$$

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

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

$$V_{max} = ((0.077707 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1803 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8906 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.1803 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8906 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.48575 \text{ kip}$$

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

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

$$M_{max} = ((0.077707 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(4.1803 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.8906 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.1803 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8906 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1803 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8906 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.7749 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.269 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.388 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.388 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.269 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023434$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.269 \text{ kip} \rightarrow 6269 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6269 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

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

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

Considering x-direction:

$V_{max} = 15.365 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.365 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.13888$$

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

Considering z-direction:

$V_{max} = 0.48575 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.48575 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.0043904$$

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:

$M_{max} = 60.395 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(60.395 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.24197$$

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

Considering z-direction:

$M_{max} = 1.7749 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.7749 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0071111$$

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

REFERENCES	CALCULATIONS	RESULTS
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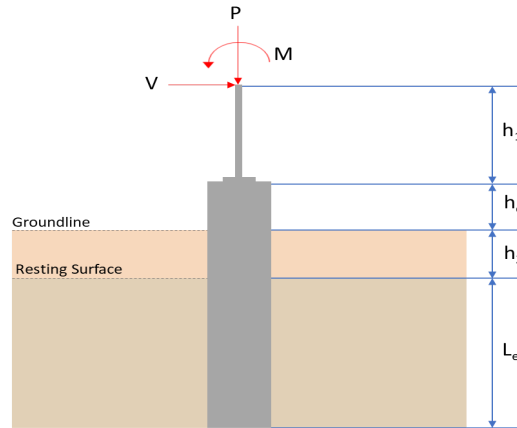
SkyCiv Foundation Design

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

$L = 8.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.468	6.269
V_x (kip)	-3.761	-6.257
V_z (kip)	-0.317	-0.488
M_x (kipft)	-1.330	-2.042
M_z (kipft)	55.518	93.228

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,

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

$$H_o = \frac{(-3.761 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59889 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.518 \text{ kipft}) + ((-3.761 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.8404 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5768 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.317 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.050478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.33 \text{ kipft}) + ((-0.317 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21178 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1788 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5768 \text{ ft}), (2.1788 \text{ ft})]$$

$$L_{e,req} = 7.577 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.577 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91842$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.468 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.27925 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.27925 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.13962$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59889 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8404 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8404 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (8.8404 \text{ kipft/ft})) + (4 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

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

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

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

$$p = \frac{0.75 \times [(4 \times (8.8404 \text{ kipft/ft})) + (3 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (8.8404 \text{ kipft/ft})) + (2 \times (-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.27937 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.8404 \text{ kipft/ft})) + ((-0.59889 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1231 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6866 \text{ ft})}{2}$$

$$p_a = 0.4265 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27937 \text{ kip/ft}^2)}{(0.4265 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65504$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1231 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90755$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = -0.050478 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.21178 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21178 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.050478 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.21178 \text{ kipft/ft})) + (4 \times (-0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.89 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.21178 \text{ kipft/ft})) + (3 \times (-0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.21178 \text{ kipft/ft})) + (2 \times (-0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = -0.0090236 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.21178 \text{ kipft/ft})) + ((-0.050478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.00062817 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.89 \text{ ft})}{2}$$

$$p_a = 0.44175 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0090236 \text{ kip/ft}^2)}{(0.44175 \text{ kip/ft}^2)}$$

$$Ratio = -0.020427$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

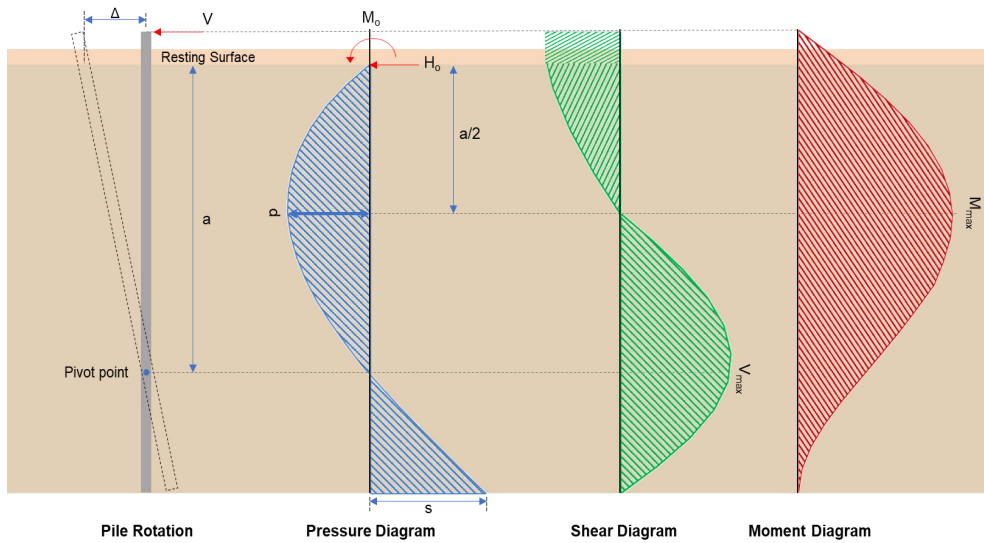
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00062817 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.00050761$$

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,

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

$$H_o = \frac{(-6.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.99634 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(93.228 \text{ kipft}) + ((-6.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.845 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.845 \text{ kipft/ft})}{(-0.99634 \text{ kip/ft})}$$

$$E = 14.9 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.845 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (14.845 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = \frac{6 \times (14.845 \text{ kipft/ft}) + (4 \times (-0.99634 \text{ kip/ft}) \times (8.25 \text{ ft}))}{\dots}$$

$$a = 5.6854 \text{ ft}$$

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

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

$$V_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6854 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6854 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.366 \text{ kip}$$

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

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

$$M_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(14.9 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.9 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6854 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 60.396 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.488 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.077707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.042 \text{ kipft}) + ((-0.488 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.32516 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.32516 \text{ kipft/ft})}{(-0.077707 \text{ kip/ft})}$$

$$E = 4.1844 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.32516 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.077707 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.32516 \text{ kipft/ft})) + (4 \times (-0.077707 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8904 \text{ ft}$$

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

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

$$V_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1844 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8904 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.1844 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8904 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.48602 \text{ kip}$$

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

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

$$M_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(4.1844 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.8904 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.1844 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.8904 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1844 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.8904 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.7761 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.269 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.388 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.388 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.269 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0023434$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.269 \text{ kip} \rightarrow 6269 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6269 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

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

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

Considering x-direction:

V_{max} = 15.366 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.366 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.13888$$

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

Considering z-direction:

$V_{max} = 0.48602 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.48602 \text{ kip})}{(110.64 \text{ kip})}$$

$$Ratio = 0.0043928$$

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:

$M_{max} = 60.396 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(60.396 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.24197$$

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

Considering z-direction:

$M_{max} = 1.7761 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.7761 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0071156$$

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

REFERENCES	CALCULATIONS	RESULTS
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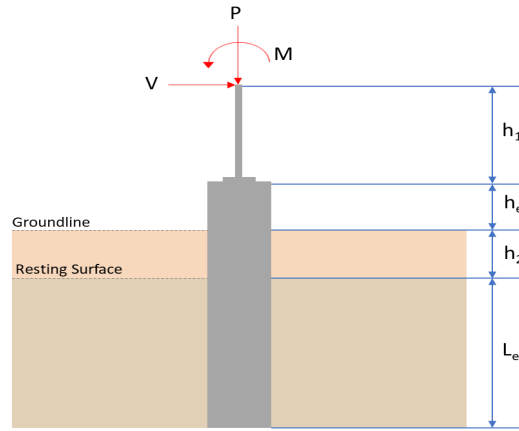
SkyCiv Foundation Design

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

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.811	8.230
V_x (kip)	-5.104	-8.491
V_z (kip)	-0.018	-0.030
M_x (kipft)	-0.067	-0.112
M_z (kipft)	75.780	127.378

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,

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

$$H_o = \frac{(-5.104 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.81274 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(75.78 \text{ kipft}) + ((-5.104 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.067 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.256 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010669 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.88818 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.256 \text{ ft}), (0.88818 \text{ ft})]$$

$$L_{e,req} = 8.256 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.256 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.91733$$

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

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

$$q = \frac{(5.811 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36319 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36319 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18159$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.81274 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.067 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.067 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.067 \text{ kipft/ft})) + (4 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2159 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.067 \text{ kipft/ft})) + (3 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.067 \text{ kipft/ft})) + (2 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.29743 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.067 \text{ kipft/ft})) + ((-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2459 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2159 \text{ ft})}{2}$$

$$p_a = 0.46619 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29743 \text{ kip/ft}^2)}{(0.46619 \text{ kip/ft}^2)}$$

$$Ratio = 0.63801$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2459 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.92286$$

Status: **PASS**
Ratio: **0.640**

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

Considering z-direction:

$H_o = -0.0028662 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010669 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010669 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.010669 \text{ kipft/ft})) + (4 \times (-0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4629 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.010669 \text{ kipft/ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 [(3 \times (0.010669 \text{ kipft/ft})) + (2 \times (-0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = -0.00056967 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.010669 \text{ kipft/ft})) + ((-0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = -0.00033027 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4629 \text{ ft})}{2}$$

$$p_a = 0.48471 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00056967 \text{ kip/ft}^2)}{(0.48471 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0011753$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

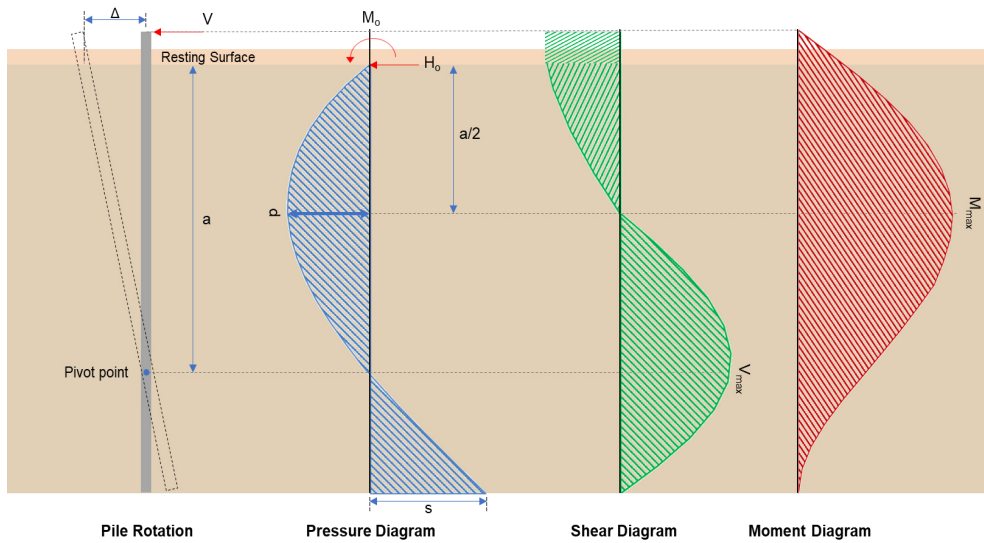
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.00033027 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.00024464$$

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,

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

$$H_o = \frac{(-8.491 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3521 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(127.38 \text{ kipft}) + ((-8.491 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.283 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(20.283 \text{ kipft/ft})}{(-1.3521 \text{ kip/ft})}$$

$$E = 15.002 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (20.283 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (20.283 \text{ kipft/ft})) + (4 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = \frac{(6 \times (20.283 \text{ kipft/ft})) + (4 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.2143 \text{ ft}$$

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

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

$$V_{max} = ((-1.3521 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2143 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2143 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 19.518 \text{ kip}$$

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

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

$$M_{max} = ((-1.3521 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(15.002 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 83.415 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.112 \text{ kipft}) + ((-0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017834 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017834 \text{ kipft/ft})}{(-0.0047771 \text{ kip/ft})}$$

$$E = 3.7333 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017834 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.017834 \text{ kipft/ft})) + (4 \times (-0.0047771 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4623 \text{ ft}$$

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

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

$$V_{max} = ((-0.0047771 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7333 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4623 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.7333 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4623 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.026794 \text{ kip}$$

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

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

$$M_{max} = ((-0.0047771 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.7333 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4623 \text{ ft})}{2 \times (9 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.7333 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4623 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7333 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4623 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.10527 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.23 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.323 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.323 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.23 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0030764$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.23 \text{ kip} \rightarrow 8230 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8230 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.58 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.58 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.58 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.58 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.81 \text{ kip}$$

Considering x-direction:

V_{max} = 19.518 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.518 \text{ kip})}{(110.81 \text{ kip})}$$

$$Ratio = 0.17614$$

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

Considering z-direction:

$V_{max} = 0.026794 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.026794 \text{ kip})}{(110.81 \text{ kip})}$$

$$Ratio = 0.0002418$$

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:

$M_{max} = 83.415 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(83.415 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.33419$$

Status: **PASS**
Ratio: **0.330**

Considering z-direction:

$M_{max} = 0.10527 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.10527 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00042176$$

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

REFERENCES	CALCULATIONS	RESULTS
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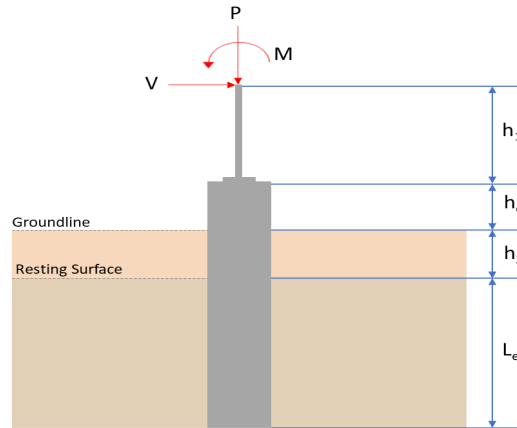
SkyCiv Foundation Design

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

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.784	8.193
V_x (kip)	-5.099	-8.499
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	76.234	128.381

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,

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

$$H_o = \frac{(-5.099 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.81194 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(76.234 \text{ kipft}) + ((-5.099 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.139 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2804 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.2804 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 8.28 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.28 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.92$$

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

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

$$q = \frac{(5.784 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.3615 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.3615 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18075$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.81194 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.139 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.139 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.81194 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.139 \text{ kipft/ft})) + (4 \times (-0.81194 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2148 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.139 \text{ kipft/ft})) + (3 \times (-0.81194 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.139 \text{ kipft/ft})) + (2 \times (-0.81194 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.30127 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.139 \text{ kipft/ft})) + ((-0.81194 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2571 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2148 \text{ ft})}{2}$$

$$p_a = 0.46611 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30127 \text{ kip/ft}^2)}{(0.46611 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64634$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

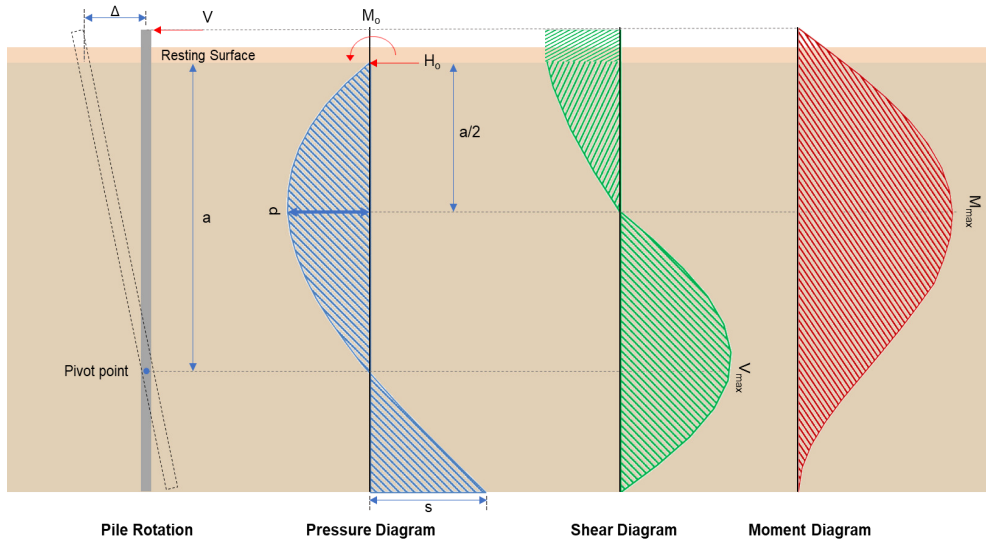
$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2571 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-8.499 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3533 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(128.38 \text{ kipft}) + ((-8.499 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.443 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(20.443 \text{ kipft/ft})}{(-1.3533 \text{ kip/ft})}$$

$$E = 15.105 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (20.443 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3533 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (20.443 \text{ kipft/ft})) + (4 \times (-1.3533 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2132 \text{ ft}$$

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

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

$$V_{max} = ((-1.3533 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.105 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2132 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (15.105 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2132 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 19.04 \text{ kip}$$

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

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

$$M_{max} = ((-1.3533 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(15.105 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2132 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.105 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2132 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (15.105 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2132 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 83.991 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

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

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

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(9 \text{ ft})} \right) \times \left(\frac{(6 \text{ ft})}{(9 \text{ ft})} - 1 \right) \times \left(\frac{(6 \text{ ft})}{(9 \text{ ft})} \right)^2$$

$$V_{max} = 0.00012582 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(6 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left(3 \times \frac{(6 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right]$$

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

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\frac{(8.193 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.324 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.324 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

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

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(8.193 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0030626$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

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

22.5.2.2

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

22.5.5.1.1

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.193 \text{ kip} \rightarrow 8193 \text{ lbf}$,

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8193 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.58 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.58 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.58 \text{ kip}$$

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

22.5.1.2

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

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

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

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

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

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

$$\phi V_n = (0.65) \times ((119.58 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.81 \text{ kip}$$

Considering x-direction:

$V_{max} = 19.647 \text{ kip}$ - Maximum shear force in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(19.647 \text{ kip})}{(110.81 \text{ kip})}$$

$$\text{Ratio} = 0.17731$$

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

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 $\phi M_{n,2}$

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

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

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

Therefore,
 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 83.991 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(83.991 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.3365$$

Status: **PASS**
Ratio: **0.340**

Considering z-direction:

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

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

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

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

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

REFERENCES	CALCULATIONS	RESULTS
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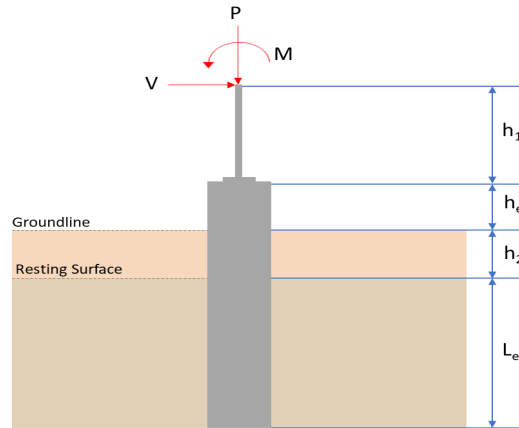
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.811	8.230
V_x (kip)	-5.104	-8.491
V_z (kip)	0.018	0.030
M_x (kipft)	0.067	0.113
M_z (kipft)	75.780	127.378

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,

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

$$H_o = \frac{(-5.104 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.81274 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(75.78 \text{ kipft}) + ((-5.104 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.067 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.256 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010669 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.009 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(8.256 \text{ ft}), (1.009 \text{ ft})]$$

$$L_{e,req} = 8.256 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(8.256 \text{ ft})}{(9 \text{ ft})}$$

$$Ratio = 0.91733$$

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

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

$$q = \frac{(5.811 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36319 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36319 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18159$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.81274 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.067 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.067 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.067 \text{ kipft/ft})) + (4 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2159 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.067 \text{ kipft/ft})) + (3 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.067 \text{ kipft/ft})) + (2 \times (-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.29743 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.067 \text{ kipft/ft})) + ((-0.81274 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2459 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2159 \text{ ft})}{2}$$

$$p_a = 0.46619 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29743 \text{ kip/ft}^2)}{(0.46619 \text{ kip/ft}^2)}$$

$$Ratio = 0.63801$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2459 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.92286$$

Status: **PASS**
Ratio: **0.640**

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

Considering z-direction:

$H_o = 0.0028662 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010669 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010669 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.010669 \text{ kipft/ft})) + (4 \times (0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4629 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010669 \text{ kipft/ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.010669 \text{ kipft/ft})) + (2 \times (0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.0015966 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010669 \text{ kipft/ft})) + ((0.0028662 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 0.0034914 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4629 \text{ ft})}{2}$$

$$p_a = 0.48471 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0015966 \text{ kip/ft}^2)}{(0.48471 \text{ kip/ft}^2)}$$

$$Ratio = 0.0032939$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

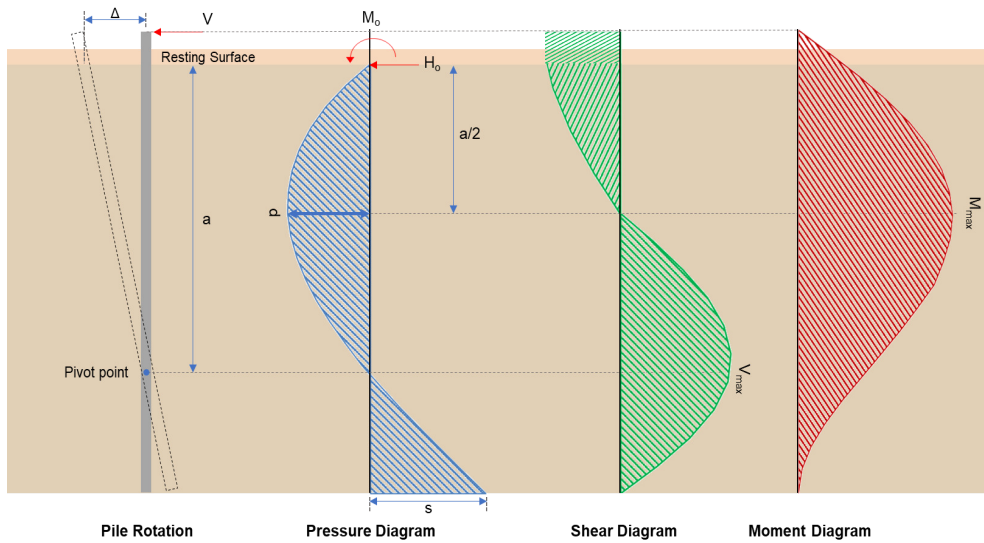
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0034914 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.0025862$$

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,

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

$$H_o = \frac{(-8.491 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3521 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(127.38 \text{ kipft}) + ((-8.491 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.283 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(20.283 \text{ kipft/ft})}{(-1.3521 \text{ kip/ft})}$$

$$E = 15.002 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (20.283 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (20.283 \text{ kipft/ft})) + (4 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = \frac{(6 \times (20.283 \text{ kipft/ft})) + (4 \times (-1.3521 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.2143 \text{ ft}$$

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

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

$$V_{max} = ((-1.3521 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2143 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2143 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 19.518 \text{ kip}$$

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

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

$$M_{max} = ((-1.3521 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(15.002 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.002 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2143 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 83.415 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.113 \text{ kipft}) + ((0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017994 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017994 \text{ kipft/ft})}{(0.0047771 \text{ kip/ft})}$$

$$E = 3.7667 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017994 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.017994 \text{ kipft/ft})) + (4 \times (0.0047771 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4608 \text{ ft}$$

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

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

$$V_{max} = ((0.0047771 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7667 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4608 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.7667 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4608 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.026917 \text{ kip}$$

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

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

$$M_{max} = ((0.0047771 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.7667 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4608 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.7667 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7667 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4608 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.10582 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.23 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.323 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.323 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.23 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0030764$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.23 \text{ kip} \rightarrow 8230 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8230 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.58 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.58 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.58 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.58 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.81 \text{ kip}$$

Considering x-direction:

V_{max} = 19.518 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.518 \text{ kip})}{(110.81 \text{ kip})}$$

$$Ratio = 0.17614$$

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

Considering z-direction:

$V_{max} = 0.026917 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.026917 \text{ kip})}{(110.81 \text{ kip})}$$

$$Ratio = 0.00024291$$

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:

$M_{max} = 83.415 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(83.415 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.33419$$

Status: **PASS**
Ratio: **0.330**

Considering z-direction:

$M_{max} = 0.10582 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.10582 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00042397$$

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