Your Project Calculations

Project Name: UnivofMNMorris-JB-RevC

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=UnivofMNMorris-JB-RevC&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=As15rfkoHQ1lsvo1U5d584hIcMDvizd8r802WBPAP5v62haNh3DrSxQz5yNyhybP

Array Specification

Product:	Beam
Unique ID:	5P-19.75-8TOP-XD-24-L-4Hx12W-KH1I
Duty Classification:	XD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	4
Number of Columns:	12
Total Number of Modules:	48
Desired Tilt Angle:	30
Front Edge Clearance:	8
Total Array Height at Tilt:	15.48 ft
Total Frame Length:	90.50 ft
Frame Weight:	5214 lbs
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	90.72 ft
Rail Length:	180.60 in
Rail Spacing:	3.78 ft
Rail Check:	PASS (56% utilized)

Support Specifications

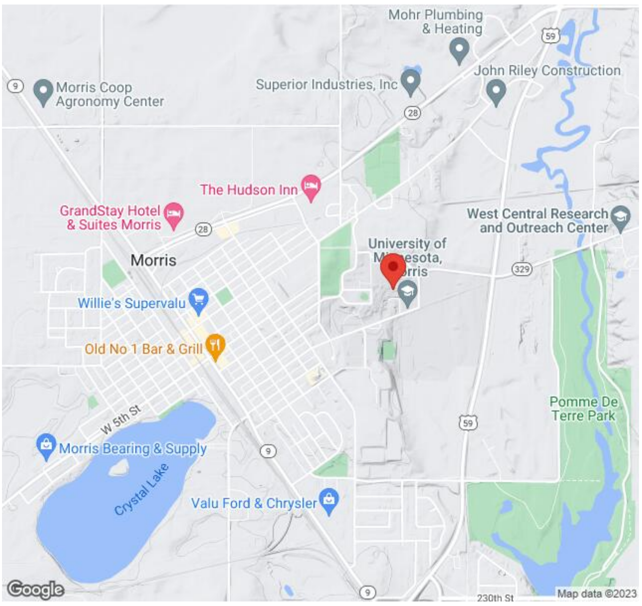
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.76 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.25 ft Pile 5: 6.00 ft
Foundation Volume:	18.222 y^3
Foundation Result:	PASSED
Mount Twist:	0.790764 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	600 E 4th St, Morris, MN 56267, USA
Wind Speed:	104 mph
Snow Load:	50 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.021993 ksf



Design Disclaimer

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

AutoDesigner Input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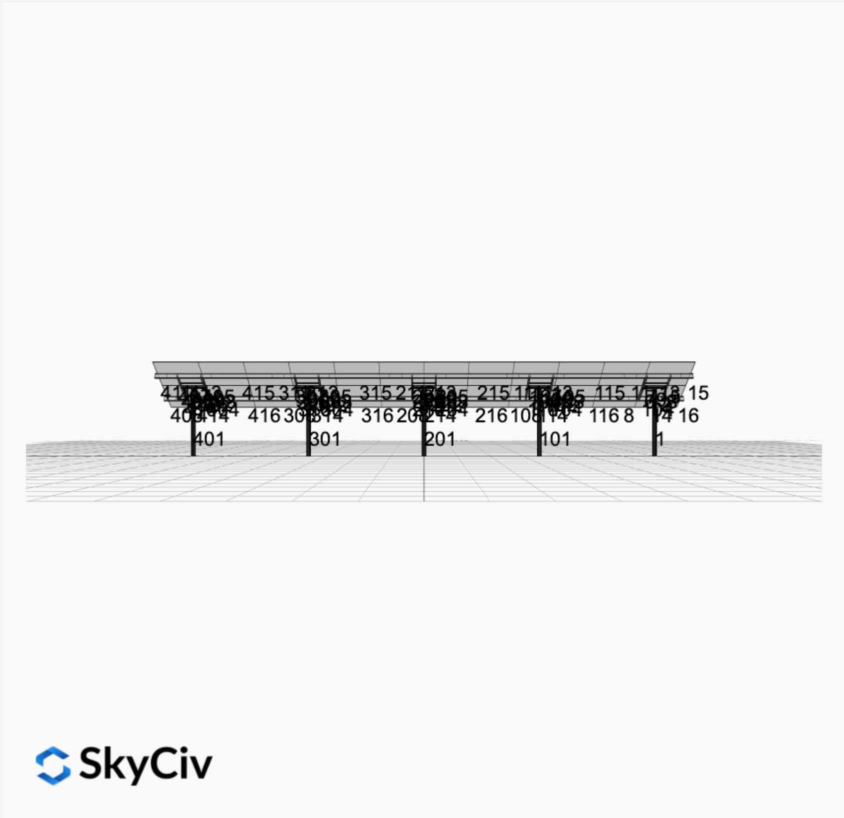
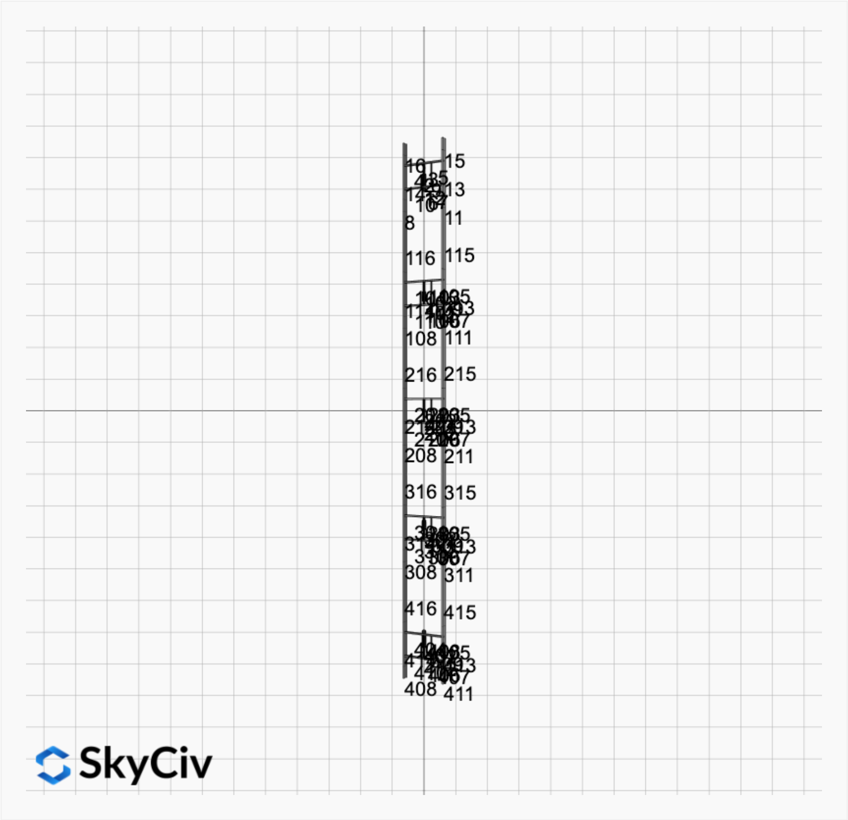
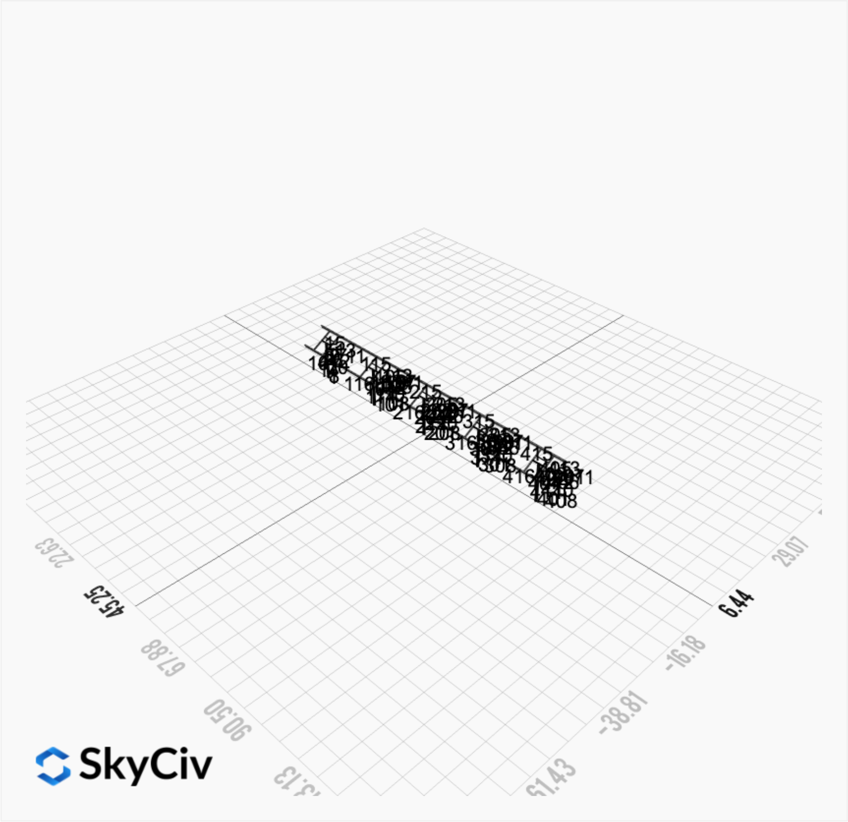
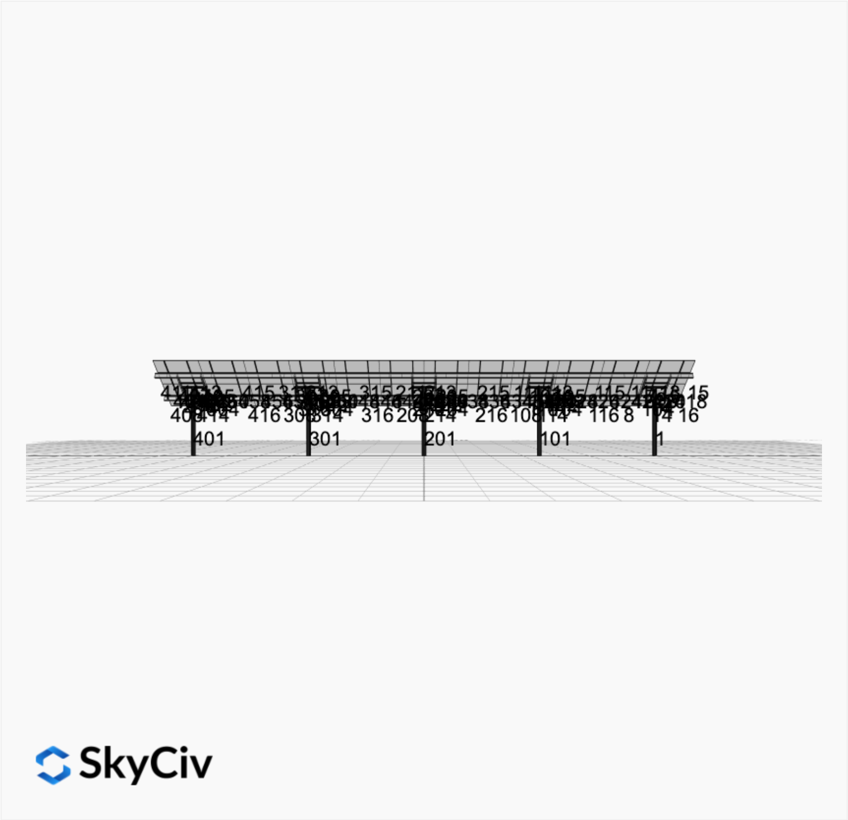
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles



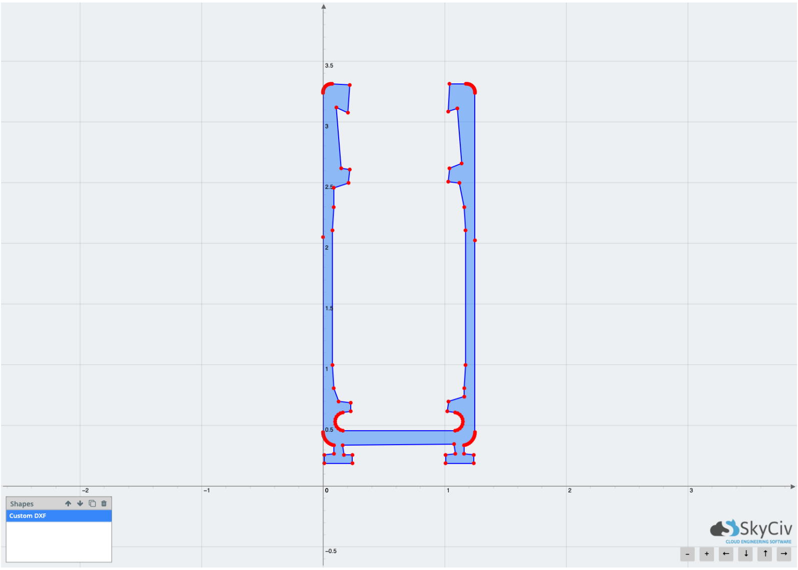




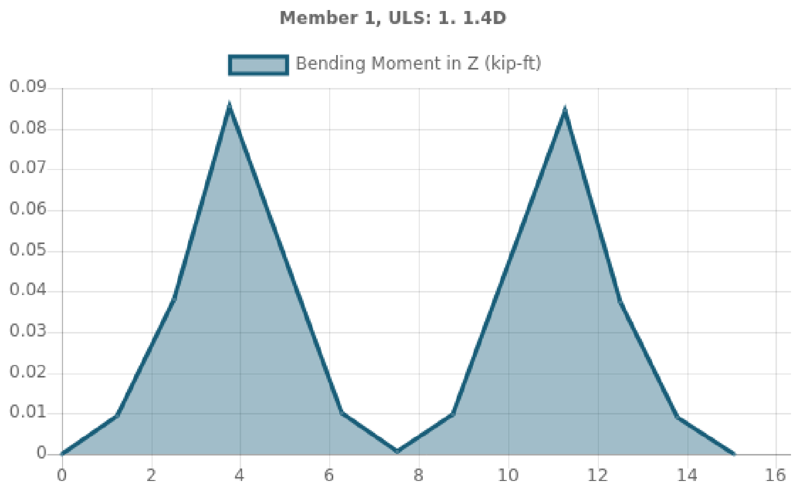


Rail Design Check

Rail Length: 15.049999999999999 ft
Additional Restraints Required: None
Tributary Width: 3.78 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0720 kip/ft
Snow (Y): -0.0416 kip/ft
Wind uplift Case A: 0.0751 kip/ft
Wind uplift Case A: 0.0751 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1043 kip/ft



Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	19.16725575	0.556	PASS
Material Yield	34.5	19.16725575	0.556	PASS
Material Strength	37	19.16725575	0.518	PASS



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	0.0231	2.0043	0.0619	0.2202	-0.0358	-0.2226
ULS: 2. D + L	0.0231	2.0043	0.0619	0.2202	-0.0358	-0.2226
ULS: 3. D + (S or Lr or R)	0.0947	6.3394	0.2550	0.9087	-0.1484	-0.9923
ULS: 3. D + (S or Lr or R)	0.0231	2.0043	0.0619	0.2202	-0.0358	-0.2226
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0768	5.2556	0.2067	0.7366	-0.1202	-0.7999
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0231	2.0043	0.0619	0.2202	-0.0358	-0.2226
ULS: 5b. D + 0.7E	0.0231	2.0043	0.0619	0.2202	-0.0358	-0.2226
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0768	5.2556	0.2067	0.7366	-0.1202	-0.7999
ULS: 8. 0.6D + 0.7E	0.0139	1.2026	0.0371	0.1321	-0.0215	-0.1335
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5960	4.7451	0.2347	0.8148	-0.4458	19.3075
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5960	4.7451	0.2347	0.8148	-0.4458	19.3075
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4083	-0.3431	-0.0831	-0.2777	0.3080	-16.4759
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1986	0.0355	-0.0784	-0.2610	0.3046	-19.7530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1375	7.3112	0.3363	1.1825	-0.4278	13.8477
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1375	7.3112	0.3363	1.1825	-0.4278	13.8477
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1157	3.4951	0.0980	0.3632	0.1376	-12.9898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9584	3.7790	0.1015	0.3757	0.1351	-15.4477
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1912	4.0599	0.1915	0.6661	-0.3433	14.4250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1912	4.0599	0.1915	0.6661	-0.3433	14.4250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0620	0.2438	-0.0469	-0.1532	0.2220	-12.4125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9047	0.5277	-0.0434	-0.1407	0.2195	-14.8704
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6052	3.9434	0.2100	0.7267	-0.4315	19.3965
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6052	3.9434	0.2100	0.7267	-0.4315	19.3965
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3990	-1.1448	-0.1078	-0.3657	0.3223	-16.3868
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1893	-0.7662	-0.1032	-0.3491	0.3189	-19.6640

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.6272
Shear X	-2.6985
Shear Z	0.5318
Moment X	1.8822
Moment Y (Twist)	0.7904
Moment Z	33.7985

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.3112
Shear X	-1.6052
Shear Z	0.3363
Moment X	1.1825
Moment Y (Twist)	0.4458
Moment Z	19.7530

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	-0.0222	2.4780	-0.0049	-0.0181	0.0108	0.2564
ULS: 2. D + L	-0.0222	2.4780	-0.0049	-0.0181	0.0108	0.2564
ULS: 3. D + (S or Lr or R)	-0.0911	8.2834	-0.0199	-0.0740	0.0443	0.9954
ULS: 3. D + (S or Lr or R)	-0.0222	2.4780	-0.0049	-0.0181	0.0108	0.2564
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0739	6.8321	-0.0161	-0.0600	0.0360	0.8106
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0222	2.4780	-0.0049	-0.0181	0.0108	0.2564
ULS: 5b. D + 0.7E	-0.0222	2.4780	-0.0049	-0.0181	0.0108	0.2564

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0739	6.8321	-0.0161	-0.0600	0.0360	0.8106
ULS: 8. 0.6D + 0.7E	-0.0133	1.4868	-0.0029	-0.0108	0.0065	0.1538
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1109	6.1542	0.0052	0.0123	-0.0428	25.0609
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1109	6.1542	0.0052	0.0123	-0.0428	25.0609
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7708	-0.6759	-0.0116	-0.0378	0.0513	-20.2990
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4504	-0.1315	-0.0223	-0.0745	0.0827	-23.9229
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6404	9.5893	-0.0086	-0.0373	-0.0042	19.4140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6404	9.5893	-0.0086	-0.0373	-0.0042	19.4140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2709	4.4667	-0.0212	-0.0748	0.0663	-14.6059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0305	4.8750	-0.0292	-0.1023	0.0899	-17.3238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5887	5.2352	0.0027	0.0047	-0.0294	18.8598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5887	5.2352	0.0027	0.0047	-0.0294	18.8598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3226	0.1126	-0.0099	-0.0329	0.0412	-15.1602
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0822	0.5209	-0.0180	-0.0604	0.0648	-17.8781
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1020	5.1630	0.0071	0.0195	-0.0471	24.9583
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1020	5.1630	0.0071	0.0195	-0.0471	24.9583
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7797	-1.6671	-0.0097	-0.0306	0.0470	-20.4016
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4592	-1.1227	-0.0204	-0.0673	0.0784	-24.0255

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	15.3222
Shear X	-3.5389
Shear Z	-0.0468
Moment X	-0.1665
Moment Y (Twist)	0.1557
Moment Z	42.8308

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.5893
Shear X	-2.1109
Shear Z	-0.0292
Moment X	-0.1023
Moment Y (Twist)	0.0899
Moment Z	25.0609

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	-0.0017	2.4312	0.0000	0.0000	0.0000	0.0811
ULS: 2. D + L	-0.0017	2.4312	0.0000	0.0000	0.0000	0.0811
ULS: 3. D + (S or Lr or R)	-0.0072	8.0916	0.0000	-0.0000	0.0001	0.2751
ULS: 3. D + (S or Lr or R)	-0.0017	2.4312	0.0000	0.0000	0.0000	0.0811
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0058	6.6765	0.0000	-0.0000	0.0001	0.2266
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0017	2.4312	0.0000	0.0000	0.0000	0.0811
ULS: 5b. D + 0.7E	-0.0017	2.4312	0.0000	0.0000	0.0000	0.0811
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0058	6.6765	0.0000	-0.0000	0.0001	0.2266
ULS: 8. 0.6D + 0.7E	-0.0010	1.4587	0.0000	0.0000	0.0000	0.0486
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0838	6.0473	0.0000	0.0000	0.0000	25.1336
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0838	6.0473	0.0000	0.0000	0.0000	25.1336
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7826	-0.6665	0.0000	0.0000	0.0000	-20.6395
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4861	-0.1624	0.0000	0.0000	0.0000	-24.6013
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5674	9.3886	0.0000	-0.0000	0.0001	19.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5674	9.3886	0.0000	-0.0000	0.0001	19.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3324	4.3532	0.0000	-0.0000	0.0001	-15.3139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1100	4.7313	0.0000	-0.0000	0.0001	-18.2852

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	-1.5633	5.1433	0.0000	0.0000	0.0000	18.8705
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5633	5.1433	0.0000	0.0000	0.0000	18.8705
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3365	0.1080	0.0000	0.0000	0.0000	-15.4594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1141	0.4860	0.0000	0.0000	0.0000	-18.4307
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0831	5.0748	0.0000	0.0000	0.0000	25.1012
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.0831	5.0748	0.0000	0.0000	0.0000	25.1012
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7832	-1.6390	0.0000	0.0000	0.0000	-20.6719
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4868	-1.1349	0.0000	0.0000	0.0000	-24.6337

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.9910
Shear X	-3.4756
Shear Z	0.0000
Moment X	-0.0002
Moment Y (Twist)	0.0006
Moment Z	42.8126

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.3886
Shear X	-2.0838
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	25.1336

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	-0.0222	2.4780	0.0049	0.0181	-0.0108	0.2564
ULS: 2. D + L	-0.0222	2.4780	0.0049	0.0181	-0.0108	0.2564
ULS: 3. D + (S or Lr or R)	-0.0911	8.2834	0.0199	0.0739	-0.0441	0.9954
ULS: 3. D + (S or Lr or R)	-0.0222	2.4780	0.0049	0.0181	-0.0108	0.2564
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0739	6.8321	0.0161	0.0599	-0.0358	0.8106
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0222	2.4780	0.0049	0.0181	-0.0108	0.2564
ULS: 5b. D + 0.7E	-0.0222	2.4780	0.0049	0.0181	-0.0108	0.2564
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0739	6.8321	0.0161	0.0599	-0.0358	0.8106
ULS: 8. 0.6D + 0.7E	-0.0133	1.4868	0.0029	0.0108	-0.0065	0.1538
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1109	6.1542	-0.0052	-0.0123	0.0428	25.0609
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1109	6.1542	-0.0052	-0.0123	0.0428	25.0609
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7708	-0.6759	0.0116	0.0378	-0.0513	-20.2990
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4504	-0.1315	0.0223	0.0745	-0.0827	-23.9229
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6404	9.5893	0.0086	0.0372	0.0044	19.4140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6404	9.5893	0.0086	0.0372	0.0044	19.4140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2709	4.4667	0.0212	0.0747	-0.0662	-14.6059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0305	4.8750	0.0292	0.1023	-0.0897	-17.3238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5887	5.2352	-0.0027	-0.0047	0.0294	18.8598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5887	5.2352	-0.0027	-0.0047	0.0294	18.8598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3226	0.1126	0.0099	0.0329	-0.0412	-15.1602
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0822	0.5209	0.0180	0.0604	-0.0647	-17.8781
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1020	5.1630	-0.0071	-0.0195	0.0471	24.9583
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1020	5.1630	-0.0071	-0.0195	0.0471	24.9583
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7797	-1.6671	0.0097	0.0306	-0.0470	-20.4016
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4592	-1.1227	0.0204	0.0673	-0.0784	-24.0255

Worst Case Reactions LRFD

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	15.3222
Shear X	-3.5389
Shear Z	0.0469
Moment X	0.1666
Moment Y (Twist)	0.1556
Moment Z	42.8309

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

Result	Value (kip, kip-ft)
Axial	9.5893
Shear X	-2.1109
Shear Z	0.0292
Moment X	0.1023
Moment Y (Twist)	0.0897
Moment Z	25.0609

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	0.0231	2.0043	-0.0619	-0.2202	0.0358	-0.2226
ULS: 2. D + L	0.0231	2.0043	-0.0619	-0.2202	0.0358	-0.2226
ULS: 3. D + (S or Lr or R)	0.0947	6.3394	-0.2550	-0.9089	0.1486	-0.9921
ULS: 3. D + (S or Lr or R)	0.0231	2.0043	-0.0619	-0.2202	0.0358	-0.2226
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0768	5.2556	-0.2067	-0.7367	0.1204	-0.7998
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0231	2.0043	-0.0619	-0.2202	0.0358	-0.2226
ULS: 5b. D + 0.7E	0.0231	2.0043	-0.0619	-0.2202	0.0358	-0.2226
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0768	5.2556	-0.2067	-0.7367	0.1204	-0.7998
ULS: 8. 0.6D + 0.7E	0.0138	1.2026	-0.0371	-0.1321	0.0215	-0.1335
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5960	4.7451	-0.2347	-0.8148	0.4458	19.3075
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5960	4.7451	-0.2347	-0.8148	0.4458	19.3075
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4083	-0.3431	0.0831	0.2776	-0.3079	-16.4759
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1986	0.0355	0.0784	0.2610	-0.3046	-19.7530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1375	7.3112	-0.3364	-1.1827	0.4279	13.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1375	7.3112	-0.3364	-1.1827	0.4279	13.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1157	3.4951	-0.0980	-0.3634	-0.1374	-12.9897
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9584	3.7790	-0.1015	-0.3758	-0.1349	-15.4476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1912	4.0599	-0.1915	-0.6661	0.3433	14.4250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1912	4.0599	-0.1915	-0.6661	0.3433	14.4250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0620	0.2438	0.0469	0.1532	-0.2220	-12.4125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9047	0.5277	0.0434	0.1407	-0.2195	-14.8704
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6052	3.9434	-0.2100	-0.7267	0.4315	19.3965
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6052	3.9434	-0.2100	-0.7267	0.4315	19.3965
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3990	-1.1448	0.1078	0.3657	-0.3222	-16.3868
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1893	-0.7662	0.1032	0.3491	-0.3189	-19.6640

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.6272
Shear X	-2.6985
Shear Z	-0.5318
Moment X	-1.8829
Moment Y (Twist)	0.7908
Moment Z	33.7990

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.3112
Shear X	-1.6052
Shear Z	-0.3364
Moment X	-1.1827
Moment Y (Twist)	0.4458
Moment Z	19.7530

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : UnivofMNMorris-JB-RevC
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **600 E 4th St, Morris, MN 56267, USA** categorized as **Exposure B** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Morever, the structure is classified as **Risk Category I**. The location is elevated at **1117.61 ft** above mean sea level.

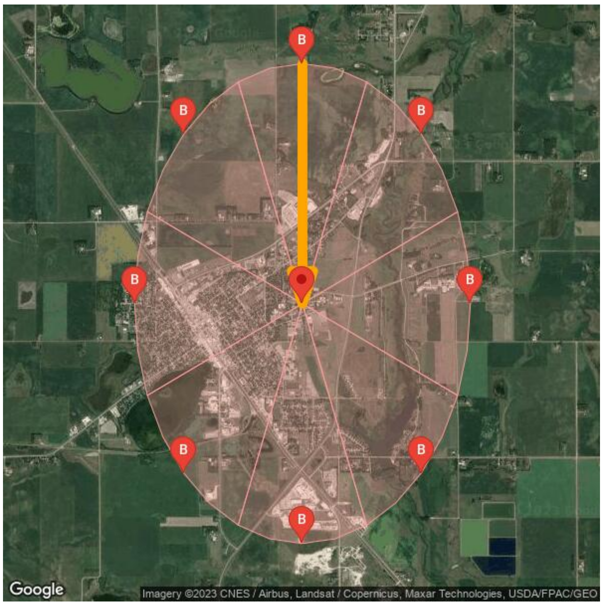


Figure 1. Site location.

Parameter	Value
Building Length, L	12.89 ft
Building Width, B	90.50 ft
Mean Roof Height, h	11.76 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.00°
Structure Type	Main Wind Force Resisting System (MWFRS)
Wind Blockage	Empty Under

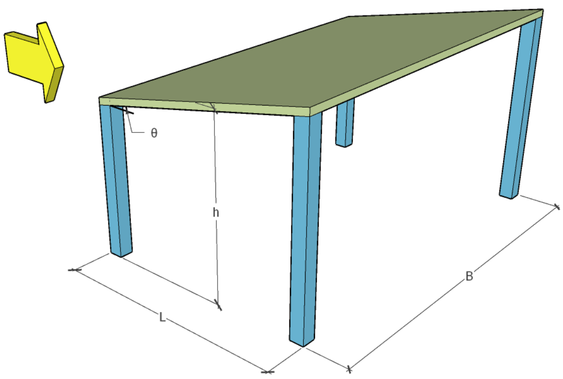


Figure 2. Building parameters.

REFERENCES	CALCULATIONS	RESULTS
Figure 26.5-1	**Basic Wind Speed, V** Wind speed for the address is **104 mph** for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.	$V = 104$ mph
Table 26.6-1	**Wind Directionality Factor, K_d** $K_d = 0.85$ - Wind Directionality Factor *For buildings*	$K_d = 0.85$
	Topographic Factor, K_{zt}	

Section 26.8.1	$K_{zt} = 1$ - Topographic Factor *For the selected wind source direction, either the terrain is relatively a flat surface or the structure is outside the local topographic zones.* **For calculating the topographic factor, the detected topography for the selected wind source direction is Flat.**	$K_{zt} = 1$

Ground Elevation Factor, K_e

Section 26.9 K_e - Ground Elevation Factor

$$K_e = e^{-0.0000362\ E}$$

$$K_e = 0.96035$$

$$\text{Where } E = \text{Site Elevation} = 1117.6 \text{ ft}$$

Section 26.9 $K_e = 0.96035$ - Ground Elevation Factor — $K_e = 0.96035$

Velocity Pressure Exposure Coefficient, K_z

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01\ (15/z_g)^{2/\alpha}$$

$$K_z = 0.57472$$

$$\text{Where } z_g = 1200$$
$$\alpha = 7$$

Level	Elevation (ft)	K_z
h	11.762	0.575

Velocity Pressure, q_h

For the selected wind source direction.

Section 26.10.2 q_h - Velocity Pressure at h

$$q_h = 0.00256\ K_z\ K_{zt}\ K_d\ K_e\ V^2$$

$$q_h = 12.99 \text{ psf}$$

$$\text{Where } K_z = \text{Velocity Pressure Exposure Coefficient} = 0.57472$$
$$K_{zt} = \text{Topographic Factor} = 1$$
$$K_d = \text{Wind Directionality Factor} = 0.85$$
$$V = \text{Basic Wind Speed} = 104 \text{ mi/h}$$
$$K_e = \text{Ground Elevation Factor} = 0.96035$$

Velocity Pressure for All Directions

Section 26.8 K_{zt} - Topographic Factor

$$K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient
For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Direction	Exposure Category	K_z @ $h = 11.762ft$	K_{zt}	K_e	V (mph)	q_h (psf)
N	B	0.575	1.000	0.960	104.000	12.990
NE	B	0.575	1.000	0.960	104.000	12.990
E	B	0.575	1.000	0.960	104.000	12.990
SE	B	0.575	1.000	0.960	104.000	12.990
S	B	0.575	1.000	0.960	104.000	12.990
SW	B	0.575	1.000	0.960	104.000	12.990
W	B	0.575	1.000	0.960	104.000	12.990
NW	B	0.575	1.000	0.960	104.000	12.990

Net Pressure Coefficients, C_N

Figure 27.3-4 to 27.3-7 The net pressure coefficients, C_N, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.

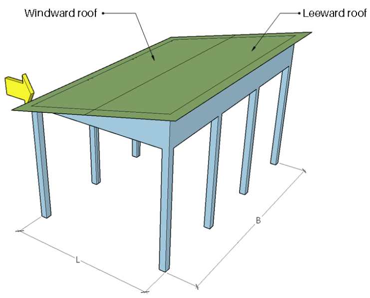
Direction	Surface	C_N Case A	C_N Case B
0	Windward	-1.800	-2.500
	Leeward		-0.500
180	Windward	2.100	2.600
	Leeward		1.000
90	≤ h from windward edge	-0.800	0.800
	h to 2h from windward edge	-0.600	0.500
	> 2h from windward edge	-0.300	0.300

Section 26.11.1	**Gust Effect Factor, G** $G = 0.85$ - Gust Effect Factor *The structure is assumed to be rigid.*	$G = 0.85$
Section 27.3.2	**Design Wind Pressures (MWFRS)** p - Design Wind Pressure *For open buildings*	

$$p = q_h \times G \times C_N$$

For Wind Pressure - 0°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
0	Windward	12.990	0.850	-1.800	-2.500	-19.875	-27.604
	Leeward	12.990	0.850		-0.500	-19.875	-5.521
180	Windward	12.990	0.850	2.100	2.600	23.187	28.708
	Leeward	12.990	0.850		1.000	23.187	11.042



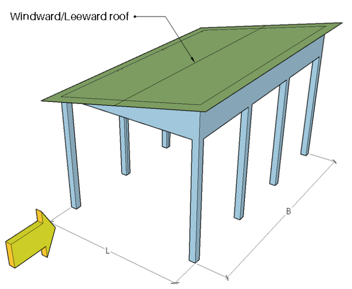
Wind along L - 0°

Service Wind Pressure - 0°/180°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	-11.925	-16.562
	Leeward	-11.925	-3.312
180	Windward	13.912	17.225
	Leeward	13.912	6.625

For Wind Pressure - 90°

Direction	Surface	q_h (psf)	G	C_N Case A	C_N Case B	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	12.990	0.850	-0.800	0.800	-8.833	8.833
	h to 2h from windward edge	12.990	0.850	-0.600	0.500	-6.625	5.521
	> 2h from windward edge	12.990	0.850	-0.300	0.300	-3.312	3.312



Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-5.300	5.300
	h to 2h from windward edge	-3.975	3.312
	> 2h from windward edge	-1.987	1.987

Section 27.3.2 *Section 28.3.5*	In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted and be reduced based on the actual solidity ratio ϕ and number of frames n - See Figure 28.3-2.	
Section 27.3.2 *Section 28.3.5*	p - Horizontal Wind Loads on Open or Partially Enclosed Buildings *For wind pressure parallel to the ridge (90°)* $$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$	
Section 27.3.2 *Section 28.3.5*	K_B - Frame Width Factor *For $L < 100ft$, $K_B = 1.8 - 0.01L$. Otherwise, $K_B = 0.8$.* $$K_B = 1.8 - 0.01 * L \leq 0.8$$ $$K_B = 1.6711$$ $$\text{Where } L = \text{Building Length} = 12.889 \text{ ft}$$	
Section 28.3.5	K_S - Shielding Factor $$K_S = 0.6 + 0.073 \times (n-1) + \left(1.25 \times \phi^{1.8}\right)$$	
Section 28.3.5	$K_S = 1$ - Shielding Factor *Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n.*	$K_S = 1$
Figure 28.3-1 *Section 28.3.5*	$(GC_{pf})_{windward} = 0.4$ *Using Zone 5 from Figure 28.3-1*	$(GC_{pf})_{windward} = 0.4$
Figure 28.3-1 *Section 28.3.5*	$(GC_{pf})_{leeward} = -0.29$ *Using Zone 6 from Figure 28.3-1*	$(GC_{pf})_{leeward} = -0.29$
Section 27.3.2 *Section 28.3.5*	p - Horizontal Wind Loads on Open or Partially Enclosed Buildings *For wind pressure parallel to the ridge (90°)* $$p = q_h\ [(GC_{pf})_{windward} - (GC_{pf})_{leeward}]\ K_B\ K_S$$ $$p = 14.978 \text{ psf}$$	

REFERENCES	CALCULATIONS	RESULTS

Snow Load Calculations based on ASCE 7-16

Design Information :

Project Name : UnivofMNMorris-JB-RevC
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **600 E 4th St, Morris, MN 56267, USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **1118 ft** above mean sea level.

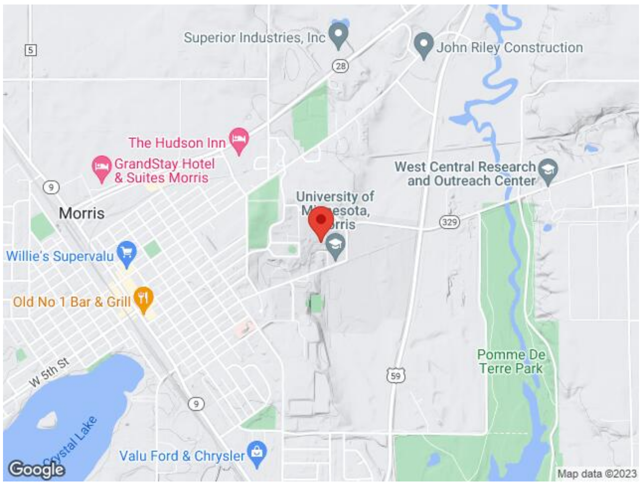


Figure 1. Site location.

Parameter	Value
Building Length, L	12.889 ft
Building Width, B	90.500 ft
Mean Roof Height, h	11.762 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	30.000°

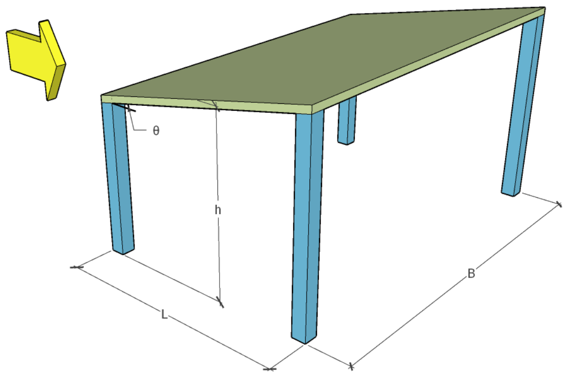


Figure 2. Building parameters.

Figure 7.6-2

S

Where roof slope is equal to $\frac{1}{S}$

$$S = \frac{1}{tan\theta}$$

$$S = 1.7321$$

$$\text{Where } \theta = \text{Angle of slope of roof} = 30\ ^\circ$$

Ground Snow Load, p_g

From Fig. 7.2-1 and Table 7.2-1
ATC/ASCE 7 Hazard Tools

$p_g = 50$ psf - Ground Snow Load

At elevation 1117.61 ft above mean sea level.

Result: $p_g = 50$ psf

Google Maps

$E = 1117.6$ ft - Ground Elevation above mean sea level

Result: $E = 1117.6$ ft

Table 7.3-1 of ASCE 7-16	**Exposure Factor, C_e** $C_e = 0.9$ - Exposure Factor *For Terrain Category **Surface Roughness B** with exposure condition specified as Fully Exposed.*	$C_e = 0.9$
Table 7.3-2 of ASCE 7-16	**Thermal Factor, C_t** $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$
Table 1.5-1 of ASCE 7-16	**Importance Factor, I_s** $I_s = 0.8$ - Importance Factor *For Risk Category I.*	$I_s = 0.8$
Section 7.3 Equation 7.3-1	**Flat Roof Snow Load, p_f** p_f - Flat Roof Snow Load $$p_f = 0.7\,C_e\,C_t\,I_s\,p_g$$ $$p_f = 30.24\text{ psf}$$ Where C_e = Exposure Factor = 0.9 C_t = Thermal Factor = 1.2 I_s = Importance Factor = 0.8 p_g = Ground Snow Load = 50 psf	
Section 7.10	**Rain-on-snow Surcharge Load, p_r** p_r - Rain-on-snow Surcharge Load *For $0 < p_g \le 20$ and $\theta < \frac{W}{50}$, $p_r = 5$ psf. Otherwise, $p_r = 0$ psf. This applies only to sloped roof (balanced) snow load case.* $$p_r = 5 : 0 < p_g \le 20, \theta < \frac{W}{50}$$ $$0 : allcases$$ $$p_r = 0\text{ psf}$$ Where p_g = Ground Snow Load = 50 psf θ = Angle of slope of roof = 30 $^\circ$ W = Horizontal distance from eave to ridge - equal to [mathin] L [mathin] for monoslope/monopitch roof. = 12.889 ft	
Equation 7.7-1	**Snow Density, γ** γ - Snow Density $$\gamma = 0.13\,p_g + 14 \le 30$$ $$\gamma = 20.5\text{ lbf/ft}^3$$ Where p_g = Ground Snow Load = 50 psf	
Table 7.3-2 of ASCE 7-16 Figure 7-2c	**Roof Slope Factor (Balanced), C_s** $C_t = 1.2$ - Thermal Factor *For Thermal Condition equal to **Unheated and open air structures**.* $\theta = 30\,^\circ$ - Angle of slope of roof $C_s = 0.72727$ - Roof Slope Factor *For roof pitch angle equal to 30.000° and Thermal Condition equal to **Unheated and open air structures**.*	$C_t = 1.2$ $\theta = 30\,^\circ$ $C_s = 0.72727$
Section 7.4	**Sloped Roof Snow Load, p_s** p_s - Sloped Roof Snow Load *Assumed to act on the horizontal projection of the surface* $$p_s = C_s\,p_f$$ $$p_s = 21.993\text{ psf}$$ Where p_f = Flat Roof Snow Load = 30.24 psf C_s = Roof Slope Factor = 0.72727	
Section 7.3.4	**Minimum Roof Snow Load, p_m** p_m - Minimum Roof Snow Load *For monoslope, hip, and gable roofs with slopes less than 15°* $$p_m = I_s\,p_g \le 20\,p_g$$ $$p_m = 16\text{ psf}$$ Where I_s = Importance Factor = 0.8 p_g = Ground Snow Load = 50 psf	

Project Details

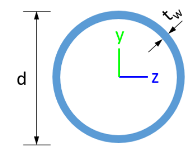
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

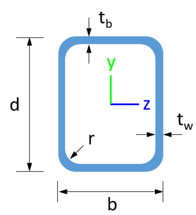
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

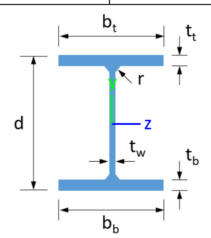
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



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



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

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	24.70	24.70	11.76	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.14,1.16,1.17,1.17,1.16,1.17,1.17,1.17,1.18,1.18,1.18,1.18,1.13,1.15,1.17,1.17,1.16,1.17	300	200	1
4	17	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.62,1.69,1.67,1.67,1.65,1.75,1.67,1.67,1.68,1.67,1.68,1.68,1.60,1.71,1.67,1.67,1.66,1.86	300	200	1
5	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
8	20	1.33	1.33	2.05	1.29,1.29,1.29,1.29,1.29,1.28,1.28,1.25,1.41,1.28,1.28,1.27,1.38,1.28,1.28,1.29,1.30,1.28,1.28,1.26,1.46,1.28,1.28,1.27,1.01	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.65,2.08,1.67,1.67,1.68,1.67,1.68,1.68,1.62,1.71,1.67,1.67,1.66,1.57	300	200	1
11	20	1.33	1.33	2.05	1.31,1.31,1.31,1.31,1.31,1.31,1.34,1.34,1.56,1.53,1.35,1.35,1.39,1.42,1.32,1.32,1.28,1.25,1.34,1.34,1.48,1.49,1.35,1.35,1.38,1.41	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.19,1.19,1.19,1.30,1.18,1.18,1.16,1.21,1.21,1.21,1.25,1.23,1.19,1.19,1.14,1.27,1.18,1.18,1.17,1.21	300	200	1
14	20	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.19,1.43,1.21,1.21,1.21,2.39,1.22,1.22,1.23,1.25,1.21,1.21,1.21,1.48,1.21,1.21,1.21,1.88	300	200	1
15	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	9	24.70	24.70	11.76	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
104	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.84,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.54	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1

108	20	1.33	1.33	2.05	2.11,2.11,2.11,2.11,2.11,2.11,2.10,2.10,2.07,2.08,2.10,2.10,2.09,1.27,2.10,2.10,2.11,2.10,2.10,2.10,2.08,2.07,2.10,2.10,2.09,1.42	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.87,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.56	300	200	1
111	20	1.33	1.33	2.05	2.08,2.08,2.08,2.08,2.08,2.08,1.97,1.97,1.35,1.47,1.89,1.89,1.69,1.66,2.06,2.06,2.11,2.36,1.97,1.97,1.45,1.52,1.87,1.87,1.73,1.68	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1
113	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.47,1.35,1.08,1.08,1.10,1.13,1.07,1.07,1.05,1.04,1.08,1.08,1.17,1.27,1.08,1.08,1.09,1.11	300	200	1
114	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.10,1.16,1.06,1.06,1.07,1.08,1.06,1.06,1.05,1.05,1.06,1.06,1.09,1.22,1.06,1.06,1.07,1.07	300	200	1
115	20	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.12,1.12,1.07,1.08,1.12,1.12,1.10,1.10,1.13,1.13,1.16,1.17,1.12,1.12,1.07,1.08,1.12,1.12,1.11,1.10	300	200	1
116	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.13,1.15,1.15,1.15,1.28,1.15,1.15,1.16,1.15,1.15,1.15,1.15,1.12,1.15,1.15,1.15,1.81	300	200	1
201	9	24.70	24.70	11.76	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
204	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.81,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.49	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
208	20	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.10,2.10,2.24,2.08,2.10,2.10,2.10,1.73,2.09,2.09,2.08,2.09,2.10,2.10,2.14,2.08,2.10,2.10,2.10,2.06	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.81,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.49	300	200	1
211	20	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.07,2.07,1.55,1.78,2.07,2.07,1.98,2.02,2.09,2.09,2.20,2.38,2.07,2.07,1.68,1.85,2.07,2.07,2.04,2.06	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.03,1.04,1.03,1.04,1.04,1.03,1.04,1.04,1.05,1.06,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.03,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04	300	200	1
214	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.10,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.14,1.04,1.04,1.04,1.01	300	200	1
215	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.12,1.14,1.15,1.15,1.14,1.14,1.16,1.16,1.17,1.18,1.15,1.15,1.13,1.14,1.15,1.15,1.14,1.15	300	200	1
216	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.18,1.15,1.16,1.16,1.16,1.14,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.15,1.16,1.16,1.16,1.43	300	200	1
301	9	24.70	24.70	11.76	-	300	200	1

302	6	1.30	1.30	2.00	-	300	200	1
303	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
304	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.87,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.71,1.67,1.67,1.66,1.56	300	200	1
305	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66	300	200	1
306	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
307	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
308	20	1.33	1.33	2.05	2.01,2.01,2.01,2.02,2.01,2.01,1.96,1.96,1.76,1.69,1.94,1.94,1.90,1.03,1.99,1.99,2.07,1.96,1.95,1.95,1.82,1.62,1.94,1.94,1.91,1.48	300	200	1
309	3	2.60	2.60	4.00	-	300	200	1
310	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.84,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.70,1.67,1.67,1.66,1.54	300	200	1
311	20	1.33	1.33	2.05	1.80,1.80,1.80,1.80,1.80,1.80,1.58,1.58,1.21,1.29,1.54,1.54,1.42,1.40,1.68,1.68,2.07,2.10,1.58,1.58,1.28,1.32,1.53,1.53,1.45,1.42	300	200	1
312	6	1.30	1.30	2.00	-	300	200	1
313	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.47,1.35,1.08,1.08,1.10,1.13,1.07,1.07,1.05,1.04,1.08,1.08,1.17,1.27,1.08,1.08,1.09,1.11	300	200	1
314	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.10,1.16,1.06,1.06,1.07,1.08,1.06,1.06,1.05,1.05,1.06,1.06,1.09,1.22,1.06,1.06,1.07,1.07	300	200	1
315	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.09,1.10,1.13,1.13,1.12,1.12,1.15,1.15,1.17,1.18,1.14,1.14,1.10,1.11,1.13,1.13,1.12,1.12	300	200	1
316	20	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.16,1.16,1.14,1.15,1.16,1.16,1.16,1.08,1.16,1.16,1.17,1.16,1.16,1.16,1.15,1.15,1.16,1.16,1.16,2.39	300	200	1
401	9	24.70	24.70	11.76	-	300	200	1
402	6	1.30	1.30	2.00	-	300	200	1
403	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.16,1.17,1.18,1.18,1.18,1.18	300	200	1
404	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.65,2.08,1.67,1.67,1.68,1.67,1.68,1.68,1.62,1.71,1.67,1.67,1.66,1.57	300	200	1
405	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
406	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.14,1.16,1.17,1.17,1.16,1.17,1.17,1.17,1.18,1.18,1.18,1.18,1.13,1.15,1.17,1.17,1.16,1.17	300	200	1
407	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.65,1.67,1.67,1.66,1.66	300	200	1
408	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
409	3	2.60	2.60	4.00	-	300	200	1
410	17	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.62,1.69,1.67,1.67,1.65,1.75,1.67,1.67,1.68,1.67,1.68,1.68,1.60,1.71,1.67,1.67,1.66,1.86	300	200	1
411	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1

412	6	1.30	1.30	2.00	-	00	00	1
413	20	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.19,1.19,1.19,1.30,1.18,1.18,1.16,1.21,1.21,1.21,1.25,1.23,1.19,1.19,1.15,1.27,1.18,1.18,1.17,1.21	300	200	1
414	20	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.19,1.43,1.21,1.21,1.21,2.40,1.22,1.22,1.23,1.25,1.22,1.22,1.21,1.47,1.21,1.21,1.21,1.89	300	200	1
415	20	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.13,1.12,1.09,1.09,1.10,1.10,1.08,1.08,1.08,1.08,1.08,1.08,1.12,1.12,1.09,1.09,1.09,1.10	300	200	1
416	20	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.08,1.08,1.90,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.10,1.08,1.08,1.08,1.34	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	179.55	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	34.84	6.46	56.26	44.91
14	159.30	116.35	36.37	6.46	56.26	44.91
15	159.30	113.66	46.90	6.46	56.26	44.91
16	159.30	113.66	46.90	6.46	56.26	44.91
101	377.97	179.55	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.78	6.46	56.26	44.91
114	159.30	116.35	32.09	6.46	56.26	44.91
115	159.30	75.13	20.61	6.46	56.26	44.91
116	159.30	75.13	21.57	6.46	56.26	44.91
201	377.97	179.55	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95

208	159.30	142.47	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.48	6.46	56.26	44.91
214	159.30	116.35	30.87	6.46	56.26	44.91
215	159.30	75.13	21.57	6.46	56.26	44.91
216	159.30	75.13	21.96	6.46	56.26	44.91
301	377.97	179.55	83.29	83.29	113.39	113.39
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	142.47	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	142.47	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	116.35	31.78	6.46	56.26	44.91
314	159.30	116.35	32.09	6.46	56.26	44.91
315	159.30	75.13	20.99	6.46	56.26	44.91
316	159.30	75.13	20.80	6.46	56.26	44.91
401	377.97	179.55	83.29	83.29	113.39	113.39
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95
407	151.65	149.10	20.17	14.14	54.12	28.95
408	159.30	113.66	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	113.66	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	116.35	35.14	6.46	56.26	44.91
414	159.30	116.35	36.37	6.46	56.26	44.91
415	159.30	75.13	20.80	6.46	56.26	44.91
416	159.30	75.13	20.80	6.46	56.26	44.91

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P, M_z, M_y)	Worst LC	KL/r	δ	Status
1	0.065	0.406	0.052	0.024	0.005	0.434	#13	0.504	Not Required	Pass
2	0.002	0.254	0.086	0.060	0.017	0.320	#21	0.054	Not Required	Pass
3	0.006	0.404	0.023	0.039	0.003	0.419	#21	0.046	Not Required	Pass
4	0.005	0.402	0.068	0.040	0.018	0.471	#21	0.082	Not Required	Pass
5	0.005	0.250	0.051	0.040	0.014	0.261	#21	0.076	Not Required	Pass
6	0.010	0.552	0.095	0.056	0.026	0.652	#21	0.046	Not Required	Pass
7	0.010	0.342	0.164	0.055	0.041	0.380	#21	0.076	Not Required	Pass
8	0.003	0.105	0.172	0.032	0.020	0.199	#24	0.102	Not Required	Pass

9	0.007	0.050	0.073	0.002	0.004	0.120	#21	0.206	Not Required	Pass
10	0.011	0.533	0.151	0.053	0.033	0.614	#21	0.082	Not Required	Pass
11	0.006	0.101	0.179	0.034	0.020	0.202	#24	0.102	Not Required	Pass
12	0.001	0.415	0.115	0.089	0.020	0.500	#21	0.054	Not Required	Pass
13	0.007	0.089	0.452	0.044	0.025	0.483	#21	0.306	Not Required	Pass
14	0.004	0.084	0.443	0.043	0.025	0.480	#24	0.204	Not Required	Pass
15	0.000	0.014	0.044	0.012	0.006	0.058	#21	Not Required	Not Required	Pass
16	0.000	0.014	0.044	0.012	0.006	0.058	#21	Not Required	Not Required	Pass
101	0.085	0.514	0.005	0.031	0.000	0.548	#13	0.504	Not Required	Pass
102	0.003	0.452	0.136	0.100	0.023	0.565	#21	0.036	Not Required	Pass
103	0.010	0.635	0.060	0.063	0.008	0.700	#21	0.046	Not Required	Pass
104	0.010	0.646	0.167	0.065	0.036	0.760	#21	0.082	Not Required	Pass
105	0.010	0.394	0.174	0.063	0.045	0.439	#21	0.076	Not Required	Pass
106	0.010	0.635	0.058	0.063	0.008	0.694	#21	0.046	Not Required	Pass
107	0.010	0.394	0.165	0.063	0.043	0.438	#21	0.076	Not Required	Pass
108	0.004	0.040	0.172	0.037	0.020	0.205	#21	0.102	Not Required	Pass
109	0.016	0.053	0.044	0.001	0.000	0.102	#21	0.206	Not Required	Pass
110	0.009	0.636	0.160	0.064	0.035	0.746	#21	0.082	Not Required	Pass
111	0.006	0.049	0.178	0.036	0.020	0.202	#21	0.102	Not Required	Pass
112	0.003	0.446	0.139	0.098	0.024	0.559	#21	0.036	Not Required	Pass
113	0.007	0.180	0.474	0.050	0.025	0.621	#21	0.306	Not Required	Pass
114	0.007	0.205	0.469	0.051	0.026	0.640	#21	0.306	Not Required	Pass
115	0.011	0.314	0.245	0.039	0.020	0.568	#21	0.507	Not Required	Pass
116	0.003	0.305	0.244	0.041	0.020	0.550	#21	0.507	Not Required	Pass
201	0.083	0.514	0.000	0.031	0.000	0.547	#13	0.504	Not Required	Pass
202	0.003	0.440	0.135	0.097	0.023	0.548	#21	0.036	Not Required	Pass
203	0.010	0.627	0.059	0.062	0.008	0.690	#21	0.046	Not Required	Pass
204	0.009	0.616	0.156	0.062	0.034	0.725	#21	0.082	Not Required	Pass
205	0.010	0.389	0.163	0.062	0.042	0.432	#21	0.076	Not Required	Pass
206	0.010	0.627	0.059	0.062	0.008	0.690	#21	0.046	Not Required	Pass
207	0.010	0.389	0.163	0.062	0.042	0.432	#21	0.076	Not Required	Pass
208	0.004	0.042	0.167	0.036	0.019	0.200	#21	0.102	Not Required	Pass
209	0.015	0.051	0.041	0.001	0.000	0.100	#21	0.206	Not Required	Pass
210	0.009	0.616	0.156	0.062	0.034	0.725	#21	0.082	Not Required	Pass
211	0.006	0.045	0.172	0.037	0.019	0.206	#21	0.102	Not Required	Pass
212	0.003	0.440	0.135	0.097	0.023	0.548	#21	0.036	Not Required	Pass
213	0.007	0.198	0.443	0.047	0.025	0.622	#21	0.306	Not Required	Pass
214	0.007	0.199	0.437	0.047	0.025	0.613	#21	0.306	Not Required	Pass
215	0.011	0.222	0.245	0.037	0.019	0.472	#21	0.507	Not Required	Pass
216	0.004	0.210	0.243	0.036	0.019	0.453	#21	0.507	Not Required	Pass
301	0.085	0.514	0.005	0.031	0.000	0.548	#13	0.504	Not Required	Pass
302	0.003	0.446	0.139	0.098	0.024	0.559	#21	0.036	Not Required	Pass
303	0.010	0.635	0.058	0.063	0.008	0.694	#21	0.046	Not Required	Pass
304	0.009	0.636	0.160	0.064	0.035	0.746	#21	0.082	Not Required	Pass
305	0.010	0.394	0.165	0.063	0.043	0.438	#21	0.076	Not Required	Pass
306	0.010	0.635	0.060	0.063	0.008	0.700	#21	0.046	Not Required	Pass
307	0.010	0.394	0.175	0.063	0.045	0.439	#21	0.076	Not Required	Pass
308	0.003	0.063	0.192	0.041	0.020	0.215	#21	0.102	Not Required	Pass
309	0.016	0.053	0.044	0.001	0.000	0.102	#21	0.206	Not Required	Pass
310	0.010	0.646	0.167	0.065	0.036	0.760	#21	0.082	Not Required	Pass
311	0.006	0.074	0.196	0.039	0.020	0.207	#21	0.102	Not Required	Pass
312	0.003	0.452	0.136	0.100	0.023	0.565	#21	0.036	Not Required	Pass
313	0.007	0.180	0.474	0.050	0.025	0.621	#21	0.306	Not Required	Pass
314	0.007	0.205	0.469	0.051	0.026	0.640	#21	0.306	Not Required	Pass

315	0.011	0.224	0.246	0.036	0.020	0.473	#21	0.507	Not Required	Pass
316	0.004	0.208	0.243	0.037	0.020	0.454	#21	0.507	Not Required	Pass
401	0.065	0.406	0.052	0.024	0.005	0.435	#13	0.504	Not Required	Pass
402	0.001	0.415	0.115	0.089	0.020	0.500	#21	0.054	Not Required	Pass
403	0.010	0.552	0.095	0.056	0.026	0.652	#21	0.046	Not Required	Pass
404	0.011	0.533	0.151	0.053	0.033	0.614	#21	0.082	Not Required	Pass
405	0.010	0.342	0.164	0.055	0.041	0.380	#21	0.076	Not Required	Pass
406	0.006	0.404	0.023	0.039	0.003	0.419	#21	0.046	Not Required	Pass
407	0.005	0.250	0.051	0.040	0.014	0.261	#21	0.076	Not Required	Pass
408	0.000	0.014	0.044	0.012	0.006	0.058	#21	Not Required	Not Required	Pass
409	0.007	0.050	0.073	0.002	0.004	0.120	#21	0.206	Not Required	Pass
410	0.005	0.402	0.068	0.040	0.018	0.471	#21	0.082	Not Required	Pass
411	0.000	0.014	0.044	0.012	0.006	0.058	#21	Not Required	Not Required	Pass
412	0.002	0.254	0.086	0.060	0.017	0.320	#21	0.054	Not Required	Pass
413	0.007	0.089	0.451	0.044	0.025	0.483	#21	0.204	Not Required	Pass
414	0.004	0.084	0.443	0.043	0.025	0.480	#24	0.306	Not Required	Pass
415	0.011	0.330	0.245	0.034	0.020	0.577	#21	0.507	Not Required	Pass
416	0.003	0.326	0.242	0.032	0.020	0.563	#21	0.507	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
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

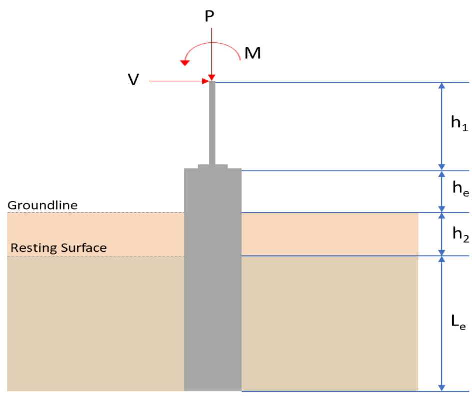
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.311	11.627
V_x (kip)	-1.605	-2.698
V_z (kip)	0.336	0.532
M_x (kipft)	1.183	1.882
M_z (kipft)	19.753	33.799

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.605\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.25557\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.753 \text{ kipft}) + ((-1.605 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1454 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.5086 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.336 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.053503 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.183 \text{ kipft}) + ((0.336 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.18838 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.8996 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.5086 \text{ ft}), (2.8996 \text{ ft})]$$

$$L_{e,req} = 5.509 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.509 \text{ ft})}{(6 \text{ ft})}$$

$$Ratio = 0.91817$$

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

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

$$q = \frac{(7.311 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.45694 \text{ kip/ft}^2$$

$q = 0.45694\ \mathrm{kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.45694\ \mathrm{kip/ft^2}\right)}{\left(2000\ \mathrm{psf}\right)}$$

$$Ratio = 0.22847$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.25557\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.1454\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1454\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.25557\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (3.1454\ \mathrm{kipft/ft})) + (4 \times (-0.25557\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.1226\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1454\ \mathrm{kipft/ft})) + (3 \times (-0.25557\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^{\ 2}}{(6\ \mathrm{ft})^2 \times [(3 \times (3.1454\ \mathrm{kipft/ft})) + (2 \times (-0.25557\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = 0.20836\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.1454\ \mathrm{kipft/ft})) + ((-0.25557\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.79289\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.1226\ \mathrm{ft})}{2}$$

$$p_a = 0.3092\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.20836\ \mathrm{kip/ft^2}\right)}{\left(0.3092\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.67386$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \text{psf/ft}) \times (6\ \text{ft})$$

$$p_s = 0.9\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.79289\ \text{kip/ft}^2\right)}{\left(0.9\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.88099$$

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = 0.053503\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.18838\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18838\ \text{kipft/ft}) \times (6\ \text{ft})) + \left(3 \times (0.053503\ \text{kip/ft}) \times (6\ \text{ft})^2\right)}{(6 \times (0.18838\ \text{kipft/ft})) + (4 \times (0.053503\ \text{kip/ft}) \times (6\ \text{ft}))}$$

$$a = 4.2659\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.18838\ \text{kipft/ft})) + (3 \times (0.053503\ \text{kip/ft}) \times (6\ \text{ft}))]^{\ 2}}{(6\ \text{ft})^2 \times [(3 \times (0.18838\ \text{kipft/ft})) + (2 \times (0.053503\ \text{kip/ft}) \times (6\ \text{ft}))]}$$

$$p = 0.050852\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.18838\ \text{kipft/ft})) + ((0.053503\ \text{kip/ft}) \times (6\ \text{ft}))]}{(6\ \text{ft})^2}$$

$$s = 0.1163\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.2659\ \text{ft})}{2}$$

$$p_a = 0.31994\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.050852\ \text{kip/ft}^2\right)}{\left(0.31994\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.15894$$

Status: **PASS**
Ratio: **0.160**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \text{psf/ft}) \times (6\ \text{ft})$$

$$p_s = 0.9\ \text{kip/ft}^2$$

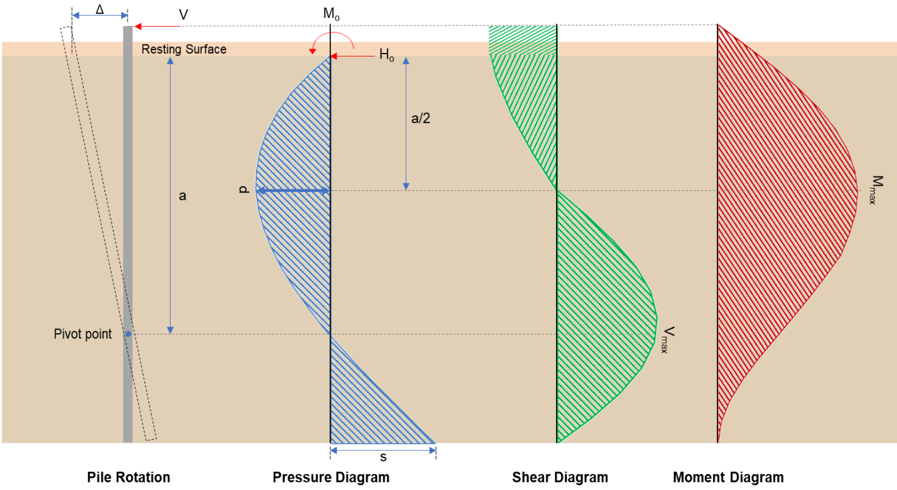
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.1163\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.12922$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.698\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42962\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.799\ \mathrm{kipft}) + ((-2.698\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.382\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.382\ \mathrm{kipft/ft})}{(-0.42962\ \mathrm{kip/ft})}$$

$$E = 12.527\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.382\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.42962\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (5.382\ \mathrm{kipft/ft})) + (4 \times (-0.42962\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.121\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.42962\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.527\ \mathrm{ft})}{(6\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.121\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.527\ \mathrm{ft})}{(6\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.121\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 7.484\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.42962\text{ kip/ft})\times(48\text{ in})\times(6\text{ ft}))\times\left[\left(\frac{(12.527\text{ ft})}{(6\text{ ft})}+\frac{(4.121\text{ ft})}{2\times(6\text{ ft})}\right) - \left[\left(\frac{4\times(12.527\text{ ft})}{(6\text{ ft})}+3\right)\times\left(\frac{(4.121\text{ ft})}{2\times(6\text{ ft})}\right)^3\right] + \left[\left(\frac{3\times(12.527\text{ ft})}{(6\text{ ft})}+2\right)\times\left(\frac{(4.121\text{ ft})}{2\times(6\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 21.514\text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.532\text{ kip})}{1.57\times(48\text{ in})}$$

$$H_o = 0.084713\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.882\text{ kipft}) + ((0.532\text{ kip})\times(0\text{ ft}))}{1.57\times(48\text{ in})}$$

$$M_o = 0.29968\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.29968\text{ kipft/ft})}{(0.084713\text{ kip/ft})}$$

$$E = 3.5376\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4\times(0.29968\text{ kipft/ft})\times(6\text{ ft})) + \left(3\times(0.084713\text{ kip/ft})\times(6\text{ ft})^2\right)}{(6\times(0.29968\text{ kipft/ft})) + (4\times(0.084713\text{ kip/ft})\times(6\text{ ft}))}$$

$$a = 4.2653\text{ ft}$$

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

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

$$V_{max} = ((0.084713\text{ kip/ft})\times(48\text{ in}))\times\left[1 - \left[3\times\left(\frac{4\times(3.5376\text{ ft})}{(6\text{ ft})}+3\right)\times\left(\frac{(4.2653\text{ ft})}{(6\text{ ft})}\right)^2\right] + \left[4\times\left(\frac{3\times(3.5376\text{ ft})}{(6\text{ ft})}+2\right)\times\left(\frac{(4.2653\text{ ft})}{(6\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.57874\text{ kip}$$

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

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

$$M_{max} = ((0.084713\text{ kip/ft})\times(48\text{ in})\times(6\text{ ft}))\times\left[\left(\frac{(3.5376\text{ ft})}{(6\text{ ft})}+\frac{(4.2653\text{ ft})}{2\times(6\text{ ft})}\right) - \left[\left(\frac{4\times(3.5376\text{ ft})}{(6\text{ ft})}+3\right)\times\left(\frac{(4.2653\text{ ft})}{2\times(6\text{ ft})}\right)^3\right] + \left[\left(\frac{3\times(3.5376\text{ ft})}{(6\text{ ft})}+2\right)\times\left(\frac{(4.2653\text{ ft})}{2\times(6\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 1.5545 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.627 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.21 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.21 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.627 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0043462$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80 \, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.627 \text{ kip} \rightarrow 11627 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11627 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.04 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.04 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.04 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.04\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 111.1\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 7.484\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.484\ \mathrm{kip})}{(111.1\ \mathrm{kip})}$$

$$Ratio = 0.06736$$

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

Considering z-direction:

$V_{max} = 0.57874\ \mathrm{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.57874\ \mathrm{kip})}{(111.1\ \mathrm{kip})}$$

$$Ratio = 0.005209$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 21.514 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(21.514 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.086192$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $M_{max} = 1.5545 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.5545 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0062278$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

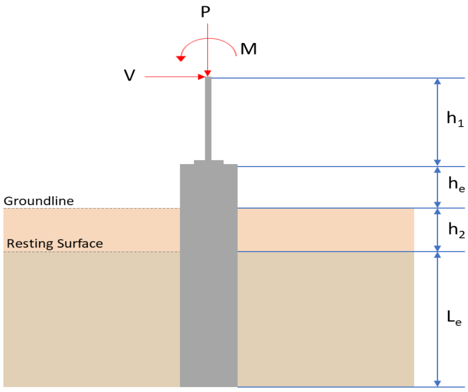
SkyCiv Foundation Design

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
$L = 6$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.311	11.627
V_x (kip)	-1.605	-2.698
V_z (kip)	-0.336	-0.532
M_x (kipft)	-1.183	-1.883
M_z (kipft)	19.753	33.799

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,

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

$$H_o = \frac{(-1.605 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25557 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.753 \text{ kipft}) + ((-1.605 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1454 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.5086 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.336 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.053503 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.183 \text{ kipft}) + ((-0.336 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.18838 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.0421 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.5086 \text{ ft}), (2.0421 \text{ ft})]$$

$$L_{e,req} = 5.509 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.509 \text{ ft})}{(6 \text{ ft})}$$

$$Ratio = 0.91817$$

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

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

$$q = \frac{(7.311 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.45694 \text{ kip/ft}^2$$

$q = 0.45694 \text{ kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.45694 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.22847$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.25557 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 3.1454 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1454 \text{ kipft/ft}) \times (6 \text{ ft})) + \left(3 \times (-0.25557 \text{ kip/ft}) \times (6 \text{ ft})^2\right)}{(6 \times (3.1454 \text{ kipft/ft})) + (4 \times (-0.25557 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1226 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1454 \text{ kipft/ft})) + (3 \times (-0.25557 \text{ kip/ft}) \times (6 \text{ ft}))]^{\;2}}{(6 \text{ ft})^2 \times [(3 \times (3.1454 \text{ kipft/ft})) + (2 \times (-0.25557 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.20836 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.1454 \text{ kipft/ft})) + ((-0.25557 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.79289 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1226 \text{ ft})}{2}$$

$$p_a = 0.3092 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.20836 \text{ kip/ft}^2\right)}{\left(0.3092 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.67386$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.79289\ \mathrm{kip/ft^2}\right)}{\left(0.9\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88099$$

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = -0.053503\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.18838\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18838\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.053503\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (0.18838\ \mathrm{kipft/ft})) + (4 \times (-0.053503\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.2659\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.18838\ \mathrm{kipft/ft})) + (3 \times (-0.053503\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^{\,2}}{(6\ \mathrm{ft})^2 \times [(3 \times (0.18838\ \mathrm{kipft/ft})) + (2 \times (-0.053503\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = -0.011895\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.18838\ \mathrm{kipft/ft})) + ((-0.053503\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.0092887\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2659\ \mathrm{ft})}{2}$$

$$p_a = 0.31994\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.011895\ \mathrm{kip/ft^2}\right)}{\left(0.31994\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.037178$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

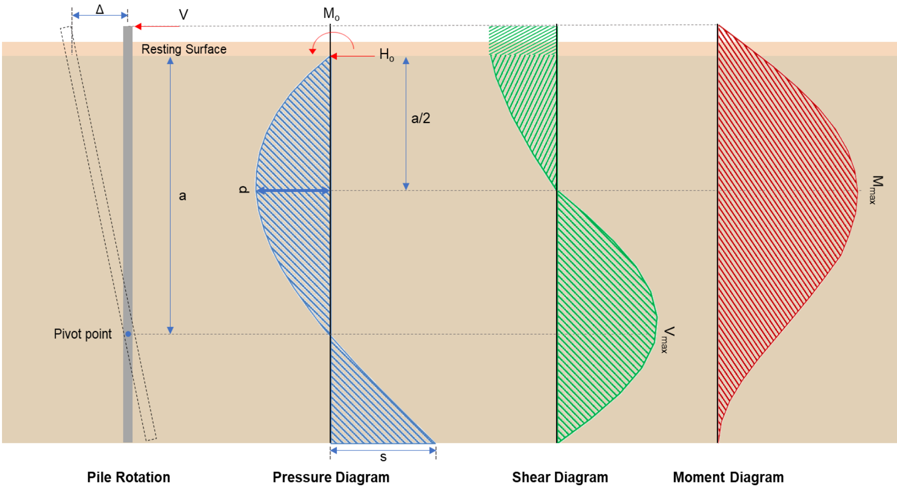
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0092887\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.010321$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.698\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.42962\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.799\ \mathrm{kipft}) + ((-2.698\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.382\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.382\ \mathrm{kipft/ft})}{(-0.42962\ \mathrm{kip/ft})}$$

$$E = 12.527\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.382\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.42962\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (5.382\ \mathrm{kipft/ft})) + (4 \times (-0.42962\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.121\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.42962\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.527\ \mathrm{ft})}{(6\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.121\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.527\ \mathrm{ft})}{(6\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.121\ \mathrm{ft})}{(6\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 7.484\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.42962 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(12.527 \text{ ft})}{(6 \text{ ft})} + \frac{(4.121 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.527 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.121 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.527 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.121 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.514 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.532 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.084713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.883 \text{ kipft}) + ((-0.532 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.29984 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.29984 \text{ kipft/ft})}{(-0.084713 \text{ kip/ft})}$$

$$E = 3.5395 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.29984 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.084713 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.29984 \text{ kipft/ft})) + (4 \times (-0.084713 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2653 \text{ ft}$$

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

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

$$V_{max} = ((-0.084713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5395 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.5395 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.57893 \text{ kip}$$

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

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

$$M_{max} = ((-0.084713 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.5395 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2653 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.5395 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5395 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.555 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.627 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.21 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.21 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.627\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0043462$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 11.627\ \mathrm{kip} \rightarrow 11627\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(11627\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 120.04\ \mathrm{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.04\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120.04\ \mathrm{kip}$$

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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.04\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.1\ \text{kip}$$

Considering x-direction:

$V_{max} = 7.484\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.484\ \text{kip})}{(111.1\ \text{kip})}$$

$$Ratio = 0.06736$$

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

Considering z-direction:

$V_{max} = 0.57893\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.57893\ \text{kip})}{(111.1\ \text{kip})}$$

$$Ratio = 0.0052107$$

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

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:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

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

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

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 21.514 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(21.514 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.086192$$

Status: **PASS**
Ratio: **0.090**

Considering z-direction:

$M_{max} = 1.555 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(1.555 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0062301$$

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

REFERENCES	CALCULATIONS	RESULTS

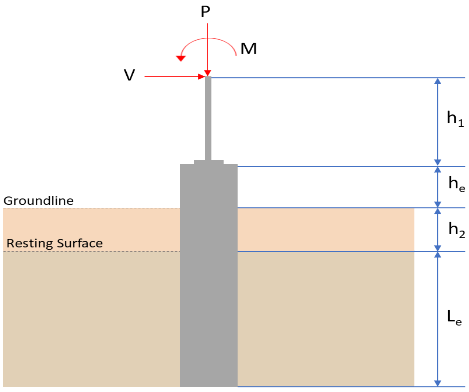
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.589	15.322
V_x (kip)	-2.111	-3.539
V_z (kip)	-0.029	-0.047
M_x (kipft)	-0.102	-0.166
M_z (kipft)	25.061	42.831

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.111\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.33615\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.061 \text{ kipft}) + ((-2.111 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.9906 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.8586 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.029 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.102 \text{ kipft}) + ((-0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016242 \text{ kipft/ft}$$

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:
$L_{e,z} = 1.0067 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8586 \text{ ft}), (1.0067 \text{ ft})]$$

$$L_{e,req} = 5.859 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.859 \text{ ft})}{(6.25 \text{ ft})}$$

$$Ratio = 0.93744$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.589 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.59931 \text{ kip/ft}^2$$

$q = 0.59931\ \mathrm{kip/ft^2}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.59931\ \mathrm{kip/ft^2}\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.29966$$

Status: **PASS**
Ratio: **0.300**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.33615\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.9906\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9906\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.33615\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (3.9906\ \mathrm{kipft/ft})) + (4 \times (-0.33615\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.302\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9906\ \mathrm{kipft/ft})) + (3 \times (-0.33615\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (3.9906\ \mathrm{kipft/ft})) + (2 \times (-0.33615\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.23057\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.9906\ \mathrm{kipft/ft})) + ((-0.33615\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.90321\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.302\ \mathrm{ft})}{2}$$

$$p_a = 0.32265\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23057\ \mathrm{kip/ft^2}\right)}{\left(0.32265\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.71462$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.90321\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.96343$$

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = -0.0046178\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.016242\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.016242\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.016242\ \mathrm{kipft/ft})) + (4 \times (-0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4491\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.016242\ \mathrm{kipft/ft})) + (3 \times (-0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\,2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.016242\ \mathrm{kipft/ft})) + (2 \times (-0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.00099718\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.016242\ \mathrm{kipft/ft})) + ((-0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.00055643\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4491\ \mathrm{ft})}{2}$$

$$p_a = 0.33368\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.00099718\ \mathrm{kip/ft^2}\right)}{\left(0.33368\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.0029884$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

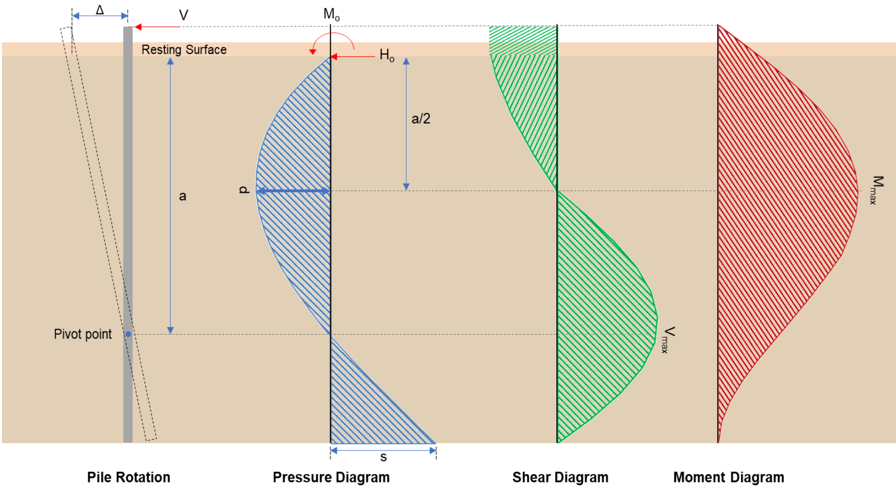
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.00055643\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.00059353$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.539\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.56354\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.831\ \mathrm{kipft}) + ((-3.539\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.8202\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8202\ \mathrm{kipft/ft})}{(-0.56354\ \mathrm{kip/ft})}$$

$$E = 12.103\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8202\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.56354\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.8202\ \mathrm{kipft/ft})) + (4 \times (-0.56354\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3001\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.56354\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.103\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3001\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.103\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3001\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.2116\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.56354\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(12.103\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.3001\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (12.103\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3001\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.103\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3001\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 27.505\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.047\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.0074841\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.166\ \text{kipft}) + ((-0.047\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.026433\ \text{kipft/ft})}{(-0.0074841\ \text{kip/ft})}$$

$$E = 3.5319\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026433\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.0074841\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.026433\ \text{kipft/ft})) + (4 \times (-0.0074841\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.4486\ \text{ft}$$

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

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

$$V_{max} = ((-0.0074841\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5319\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.4486\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.5319\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.4486\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.049844\ \text{kip}$$

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

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

$$M_{max} = ((-0.0074841\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(3.5319\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.4486\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.5319\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.4486\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.5319\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.4486\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.13905 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(15.322 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.087 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.087 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(15.322 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0057275$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80 \, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.322 \text{ kip} \rightarrow 15322 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(15322 \text{ lbf})}{6 \times \left(2304 \text{ in}^2 \right)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.53 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min \left[(296.21 \text{ kip}), (120.53 \text{ kip}), (348.89 \text{ kip}) \right]$$

$$V_c = 120.53 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.53\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 111.42\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 9.2116\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.2116\ \mathrm{kip})}{(111.42\ \mathrm{kip})}$$

$$Ratio = 0.082671$$

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

Considering z-direction:

$V_{max} = 0.049844\ \mathrm{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.049844\ \mathrm{kip})}{(111.42\ \mathrm{kip})}$$

$$Ratio = 0.00044734$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\ \mathrm{kipft}), (2121.6\ \mathrm{kipft})]$$ $$\phi M_n = 249.6\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 27.505\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(27.505\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.1102$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $M_{max} = 0.13905\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.13905\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.00055708$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

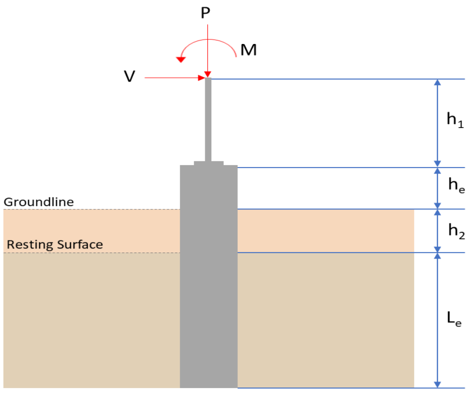
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.389	14.991
V_x (kip)	-2.084	-3.476
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	25.134	42.813

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.084\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.33185\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.134\ \text{kipft}) + ((-2.084\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.0022\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8784\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(5.8784\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 5.878\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.878\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.94048$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.389\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.58681\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.58681\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.29341$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.33185\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.0022\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0022\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.33185\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (4.0022\ \mathrm{kipft/ft})) + (4 \times (-0.33185\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3004\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0022\ \mathrm{kipft/ft})) + (3 \times (-0.33185\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (4.0022\ \mathrm{kipft/ft})) + (2 \times (-0.33185\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.23401\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.0022\ \mathrm{kipft/ft})) + ((-0.33185\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.91091\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.3004\ \mathrm{ft})}{2}$$

$$p_a = 0.32253\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23401\ \mathrm{kip/ft^2}\right)}{\left(0.32253\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.72555$$

Status: **PASS**
Ratio: **0.730**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

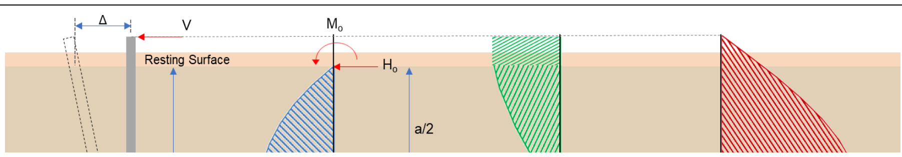
$Ratio$ - Lateral soil capacity

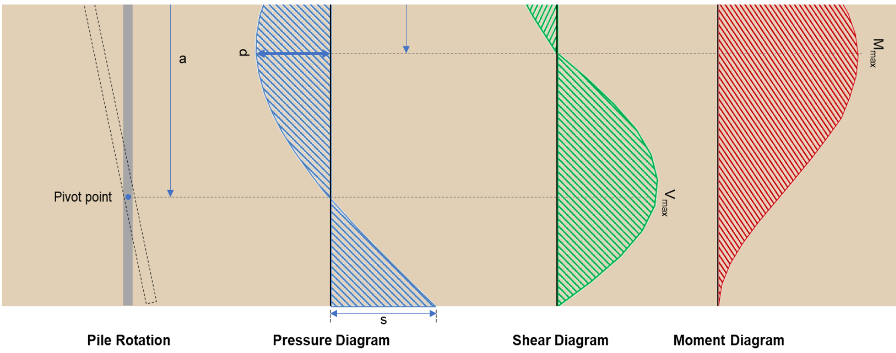
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.91091\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.97164$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.476\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.5535\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.813\text{ kipft}) + ((-3.476\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 6.8174\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8174\text{ kipft/ft})}{(-0.5535\text{ kip/ft})}$$

$$E = 12.317\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8174\text{ kipft/ft}) \times (6.25\text{ ft})) + \left(3 \times (-0.5535\text{ kip/ft}) \times (6.25\text{ ft})^2\right)}{(6 \times (6.8174\text{ kipft/ft})) + (4 \times (-0.5535\text{ kip/ft}) \times (6.25\text{ ft}))}$$

$$a = 4.2983\text{ ft}$$

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

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

$$V_{max} = ((-0.5535\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.317\text{ ft})}{(6.25\text{ ft})} + 3\right) \times \left(\frac{(4.2983\text{ ft})}{(6.25\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.317\text{ ft})}{(6.25\text{ ft})} + 2\right) \times \left(\frac{(4.2983\text{ ft})}{(6.25\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 9.1821\text{ kip}$$

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

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

$$M_{max} = ((-0.5535\text{ kip/ft}) \times (48\text{ in}) \times (6.25\text{ ft})) \times \left[\left(\frac{(12.317\text{ ft})}{(6.25\text{ ft})} + \frac{(4.2983\text{ ft})}{2 \times (6.25\text{ ft})}\right) - \left[\left(\frac{4 \times (12.317\text{ ft})}{(6.25\text{ ft})} + 3\right) \times \left(\frac{(4.2983\text{ ft})}{2 \times (6.25\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.317\text{ ft})}{(6.25\text{ ft})} + 2\right) \times \left(\frac{(4.2983\text{ ft})}{2 \times (6.25\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 27.435 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(14.991 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.098 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.098 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(14.991\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0056037$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \to 2500\ \mathrm{psi}$, $P = 14.991\ \mathrm{kip} \to 14991\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(14991\ \mathrm{lbf})}{6 \times \left(2304\ \mathrm{in}^2\right)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 120.48\ \mathrm{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.48\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120.48\ \mathrm{kip}$$

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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.48\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.4\ \text{kip}$$

Considering x-direction:

$V_{max} = 9.1821\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.1821\ \text{kip})}{(111.4\ \text{kip})}$$

$$Ratio = 0.082427$$

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

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:
$\phi M_{n,1}$

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

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

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

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = \phi\ 0.85\ f_{ck}\ S_m$$

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 27.435\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(27.435\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.10992$$

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

REFERENCES	CALCULATIONS	RESULTS

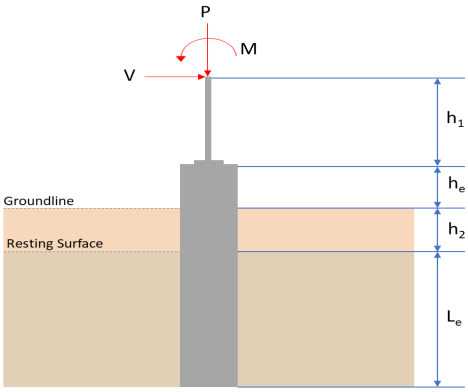
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.589	15.322
V_x (kip)	-2.111	-3.539
V_z (kip)	0.029	0.047
M_x (kipft)	0.102	0.167
M_z (kipft)	25.061	42.831

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.111\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.33615\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.061\ \mathrm{kipft}) + ((-2.111\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 3.9906\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 5.8586\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.029\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.0046178\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.102\ \mathrm{kipft}) + ((0.029\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.016242\ \mathrm{kipft/ft}$$

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:
$L_{e,z} = 1.1757\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8586\ \mathrm{ft}), (1.1757\ \mathrm{ft})]$$

$$L_{e,req} = 5.859\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.25\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.859\ \mathrm{ft})}{(6.25\ \mathrm{ft})}$$

$$Ratio = 0.93744$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.589\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.59931\ \mathrm{kip/ft}^2$$

$q = 0.59931 \text{ kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.59931 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.29966$$

Status: **PASS**
Ratio: **0.300**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.33615 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 3.9906 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9906 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.33615 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (3.9906 \text{ kipft/ft})) + (4 \times (-0.33615 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.302 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9906 \text{ kipft/ft})) + (3 \times (-0.33615 \text{ kip/ft}) \times (6.25 \text{ ft}))]^{\ 2}}{(6.25 \text{ ft})^2 \times [(3 \times (3.9906 \text{ kipft/ft})) + (2 \times (-0.33615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.23057 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.9906 \text{ kipft/ft})) + ((-0.33615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.90321 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.302 \text{ ft})}{2}$$

$$p_a = 0.32265 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.23057 \text{ kip/ft}^2)}{(0.32265 \text{ kip/ft}^2)}$$

$$Ratio = 0.71462$$

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

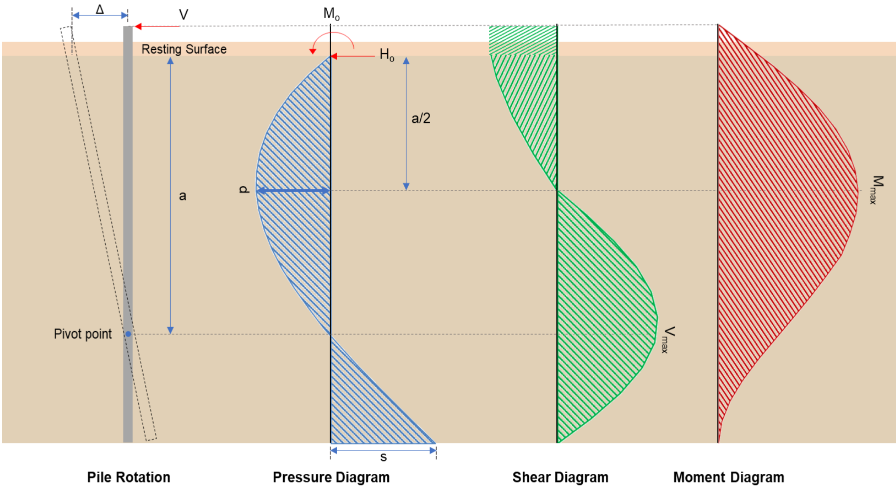
p_s - Allowable lateral soil pressure at depth L_e,

	$$p_s = R\, L_e$$ $$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$ $$p_s = 0.9375\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$ $$Ratio = \frac{(0.90321\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$ $$Ratio = 0.96343$$	Status: PASS Ratio: 0.960	
	Considering z-direction: $H_o = 0.0046178\ \mathrm{kip/ft}$ - Lateral force per length of pile, $M_o = 0.016242\ \mathrm{kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $$a = \frac{(4\, M_o\, L_e) + \left(3\, H_o\, L_e^2\right)}{(6\, M_o) + (4\, H_o\, L_e)}$$ $$a = \frac{(4 \times (0.016242\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.016242\ \mathrm{kipft/ft})) + (4 \times (0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$ $$a = 4.4491\ \mathrm{ft}$$ p - Earth pressure against the pile at distance a/2 from resting surface, $$p = \frac{0.75\, [(4\, M_o) + (3\, H_o\, L_e)]^2}{L_e^2\, [(3\, M_o) + (2\, H_o\, L_e)]}$$ $$p = \frac{0.75 \times [(4 \times (0.016242\ \mathrm{kipft/ft})) + (3 \times (0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.016242\ \mathrm{kipft/ft})) + (2 \times (0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$ $$p = 0.0041427\ \mathrm{kip/ft^2}$$ s - Earth pressure against the pile at distance L_e, $$s = \frac{6\, [(2\, M_o) + (H_o\, L_e)]}{L_e^2}$$ $$s = \frac{6 \times [(2 \times (0.016242\ \mathrm{kipft/ft})) + ((0.0046178\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$ $$s = 0.0094227\ \mathrm{kip/ft^2}$$ **Check lateral soil pressure capacity:** p_a - Allowable lateral soil pressure at depth a/2, $$p_a = R\, \frac{a}{2}$$ $$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4491\ \mathrm{ft})}{2}$$ $$p_a = 0.33368\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{(0.0041427\ \mathrm{kip/ft^2})}{(0.33368\ \mathrm{kip/ft^2})}$$ $$Ratio = 0.012415$$	Status: PASS Ratio: 0.010	
	p_s - Allowable lateral soil pressure at depth L_e, $$p_s = R\, L_e$$ $$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$ $$p_s = 0.9375\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$		

$$Ratio = \frac{(0.0094227\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.010051$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.539\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.56354\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.831\ \mathrm{kipft}) + ((-3.539\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.8202\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8202\ \mathrm{kipft/ft})}{(-0.56354\ \mathrm{kip/ft})}$$

$$E = 12.103\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8202\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.56354\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (6.8202\ \mathrm{kipft/ft})) + (4 \times (-0.56354\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3001\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.56354\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.103\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.3001\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.103\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.3001\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.2116\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.56354 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(12.103 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3001 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.103 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3001 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.103 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3001 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.505 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.047 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0074841 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.167 \text{ kipft}) + ((0.047 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026592 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.026592 \text{ kipft/ft})}{(0.0074841 \text{ kip/ft})}$$

$$E = 3.5532 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026592 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (0.0074841 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.026592 \text{ kipft/ft})) + (4 \times (0.0074841 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4478 \text{ ft}$$

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

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

$$V_{max} = ((0.0074841 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5532 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4478 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.5532 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4478 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.050023 \text{ kip}$$

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

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

$$M_{max} = ((0.0074841 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.5532 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4478 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.5532 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4478 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5532 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4478 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1396 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(15.322 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.087 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.087 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(15.322\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0057275$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 15.322\ \mathrm{kip} \rightarrow 15322\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(15322\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 120.53\ \mathrm{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.53\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120.53\ \mathrm{kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.53\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 111.42\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 9.2116\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.2116\ \mathrm{kip})}{(111.42\ \mathrm{kip})}$$

$$Ratio = 0.082671$$

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

Considering z-direction:

$V_{max} = 0.050023\ \mathrm{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.050023\ \mathrm{kip})}{(111.42\ \mathrm{kip})}$$

$$Ratio = 0.00044894$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 27.505 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(27.505 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.1102$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $M_{max} = 0.1396 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.1396 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.00055931$$	Status: **PASS** Ratio: **0.000**