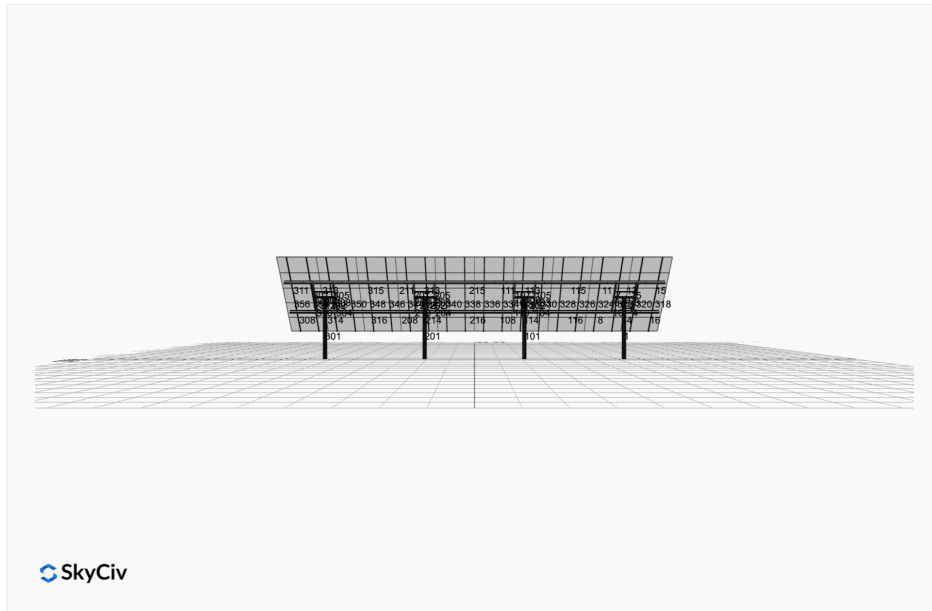


Project Name: MTSOLAR_A576EK26A31K **Date:** Thu Nov 21 2024
Location: New Richmond, WI 54017, USA **Number of Modules:** 50
Unique ID: 4P-19.75-8TOP-HD-45-L-5Hx10W-D833 **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	75.54 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

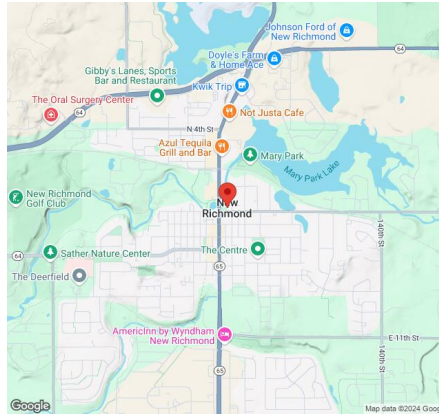
MT Solar Bill of Materials (4P-19.75-8TOP-HD-45-L-5Hx10W-D833)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	6
MTS-HD-Splice-57	57IN HD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	10

Rail Bill of Materials

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

Site Details:



Site Address: New Richmond, WI 54017, USA

Array Specification

Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.65in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	19.41 ft
Total Frame Length:	74.25 ft
Frame Weight:	5523 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	75.54 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	12.21 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.25 ft Pile 2: 7.50 ft Pile 3: 7.50 ft Pile 4: 7.25 ft
Foundation Volume:	17.481 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	New Richmond, WI 54017, USA
Wind Speed:	101 mph

Snow Load:

50 psf

Design Disclaimer

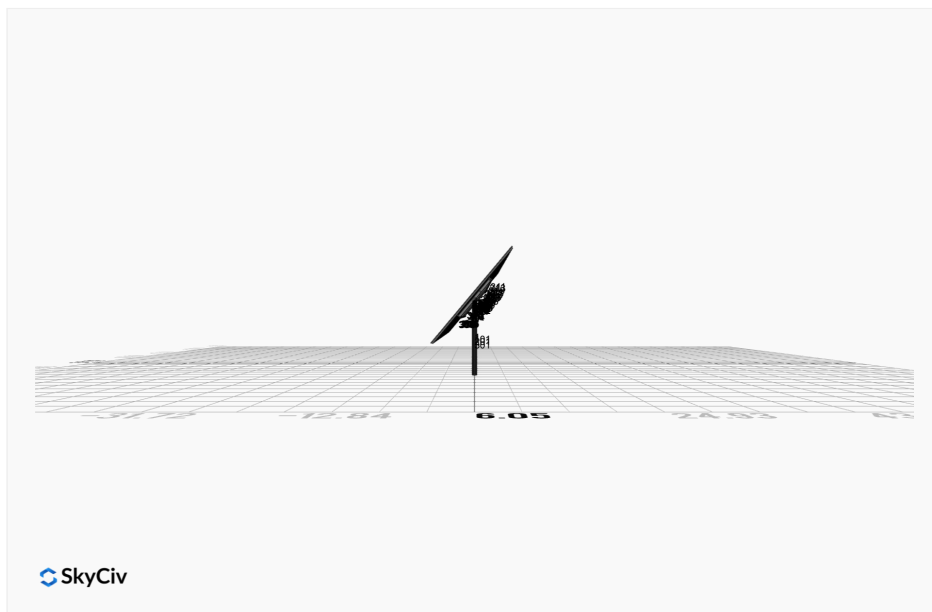
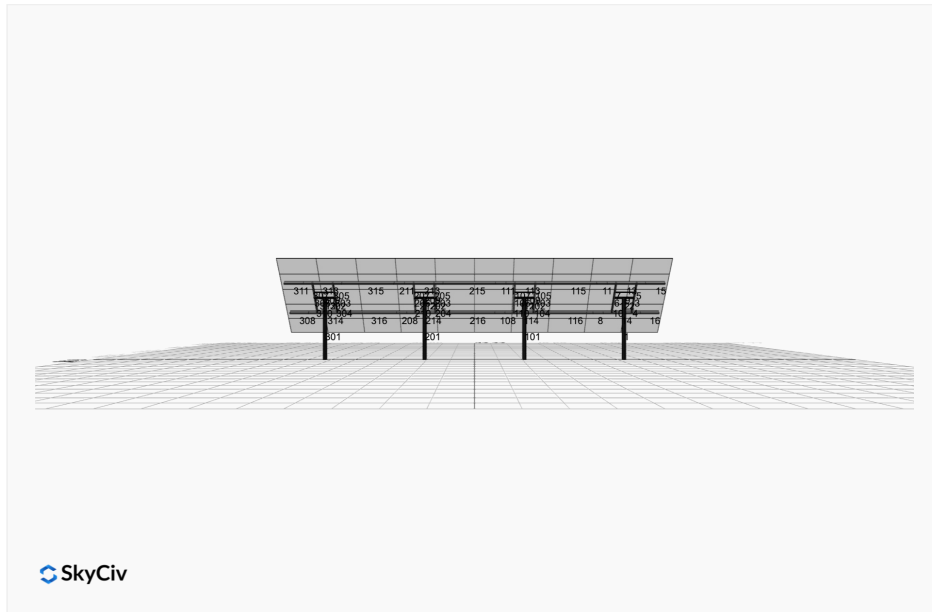
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

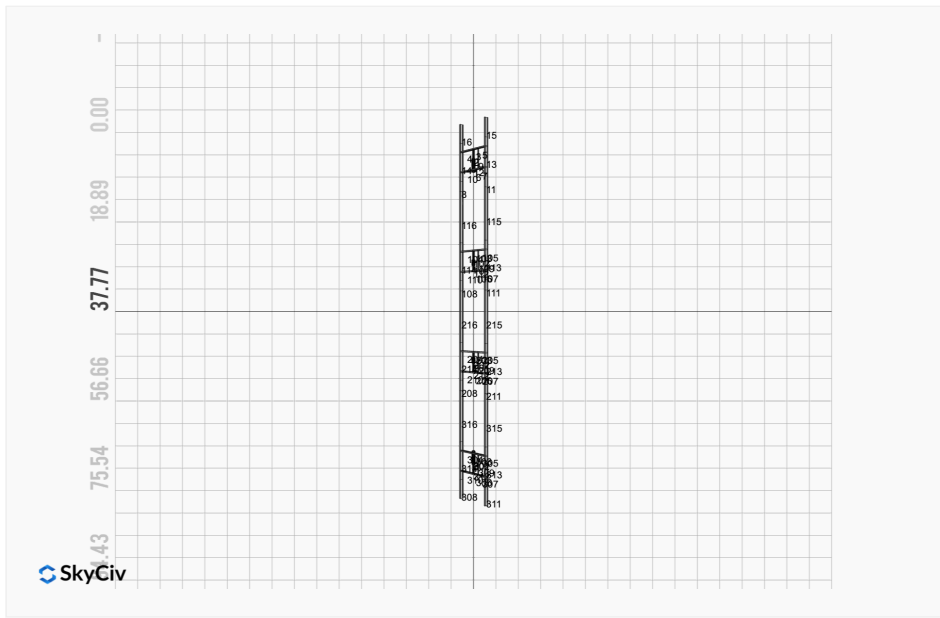
AutoDesigner Input

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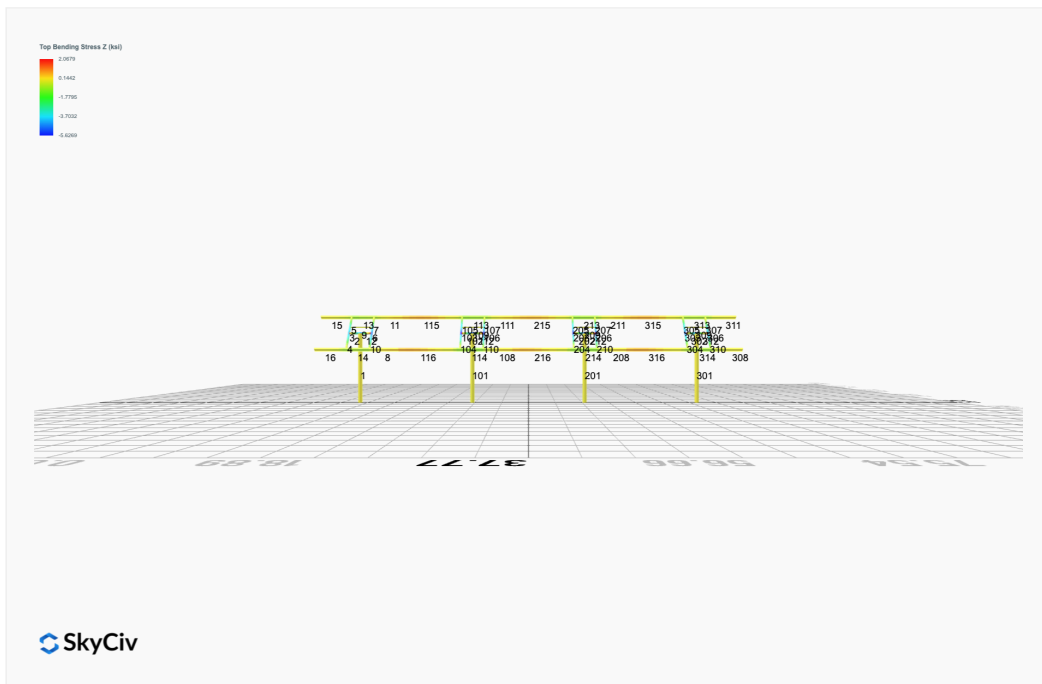
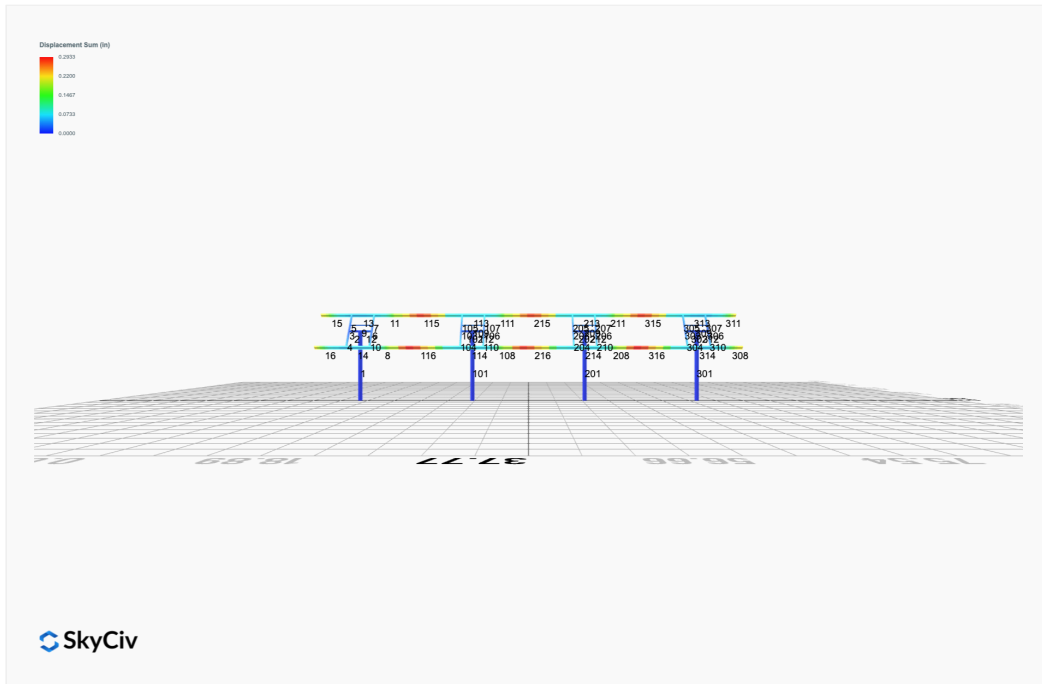
Design Notes:

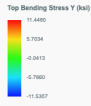
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



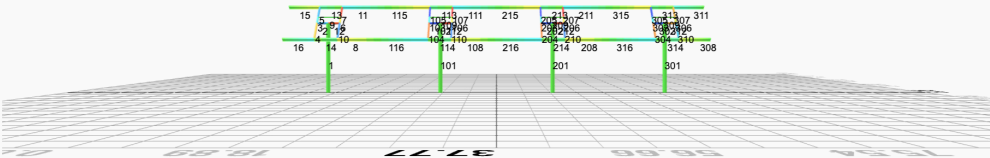
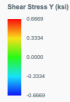


FEM Results (Envelope Worst Case for each member)

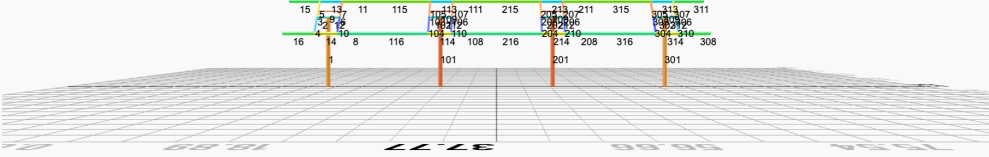
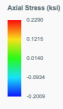




SkyCiv



SkyCiv



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	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 2. D + L	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 3. D + (S or Lr or R)	0.0196	4.8542	0.0721	0.2622	-0.0257	-0.2022
ULS: 3. D + (S or Lr or R)	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0169	4.2792	0.0620	0.2254	-0.0221	-0.1716
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 5b. D + 0.7E	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0169	4.2792	0.0620	0.2254	-0.0221	-0.1716
ULS: 8. 0.6D + 0.7E	0.0052	1.5326	0.0190	0.0691	-0.0066	-0.0479
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2950	5.3047	0.1188	0.4017	-0.5518	40.7949
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3102	-0.1952	-0.0534	-0.1645	0.5181	-40.0576
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4609	6.3420	0.1273	0.4403	-0.4277	30.4844
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0169	4.2792	0.0620	0.2254	-0.0221	-0.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4931	2.2171	-0.0018	0.0157	0.3748	-30.1550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0169	4.2792	0.0620	0.2254	-0.0221	-0.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4691	4.6171	0.0970	0.3300	-0.4166	30.5763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4848	0.4921	-0.0321	-0.0946	0.3858	-30.0631
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0086	2.5543	0.0317	0.1151	-0.0111	-0.0798
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2985	4.2830	0.1061	0.3556	-0.5474	40.8269
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0052	1.5326	0.0190	0.0691	-0.0066	-0.0479
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3068	-1.2170	-0.0661	-0.2106	0.5225	-40.0257
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0052	1.5326	0.0190	0.0691	-0.0066	-0.0479

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	9.0374
Shear X	-5.5199
Shear Z	0.2047
Moment X	0.6939
Moment Y (Twist)	0.9291
Moment Z	68.7374

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	6.3420
Shear X	-3.3102
Shear Z	0.1273
Moment X	0.4403
Moment Y (Twist)	0.5518
Moment Z	40.8269

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	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 2. D + L	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 3. D + (S or Lr or R)	-0.0196	5.4563	-0.0014	-0.0056	0.0062	0.2615
ULS: 3. D + (S or Lr or R)	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0169	4.7971	-0.0012	-0.0048	0.0054	0.2270

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 5b. D + 0.7E	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0169	4.7971	-0.0012	-0.0048	0.0054	0.2270
ULS: 8. 0.6D + 0.7E	-0.0052	1.6917	-0.0004	-0.0015	0.0016	0.0740
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7544	5.9843	0.0151	0.0479	-0.1180	46.2987
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7392	-0.3461	-0.0152	-0.0490	0.1158	-44.9616
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8262	7.1707	0.0106	0.0329	-0.0852	34.8585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0169	4.7971	-0.0012	-0.0048	0.0054	0.2270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7940	2.4228	-0.0122	-0.0397	0.0902	-33.5868
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0169	4.7971	-0.0012	-0.0048	0.0054	0.2270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8180	5.1931	0.0112	0.0353	-0.0878	34.7549
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8023	0.4453	-0.0116	-0.0373	0.0875	-33.6904
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0086	2.8196	-0.0006	-0.0025	0.0027	0.1234
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7510	4.8565	0.0154	0.0489	-0.1191	46.2493
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0052	1.6917	-0.0004	-0.0015	0.0016	0.0740
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7427	-1.4739	-0.0150	-0.0480	0.1147	-45.0110
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0052	1.6917	-0.0004	-0.0015	0.0016	0.0740

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.2392
Shear X	-6.2572
Shear Z	0.0262
Moment X	-0.0843
Moment Y (Twist)	0.2021
Moment Z	78.1009

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	7.1707
Shear X	-3.7544
Shear Z	0.0154
Moment X	-0.0490
Moment Y (Twist)	0.1191
Moment Z	46.2987

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	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 2. D + L	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 3. D + (S or Lr or R)	-0.0196	5.4563	0.0014	0.0056	-0.0062	0.2615
ULS: 3. D + (S or Lr or R)	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0169	4.7971	0.0012	0.0048	-0.0053	0.2270
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 5b. D + 0.7E	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0169	4.7971	0.0012	0.0048	-0.0053	0.2270
ULS: 8. 0.6D + 0.7E	-0.0052	1.6917	0.0004	0.0015	-0.0016	0.0740
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7544	5.9843	-0.0151	-0.0479	0.1180	46.2987
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7392	-0.3461	0.0152	0.0490	-0.1158	-44.9616
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8262	7.1707	-0.0106	-0.0330	0.0852	34.8585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0169	4.7971	0.0012	0.0048	-0.0053	0.2270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7940	2.4228	0.0122	0.0397	-0.0901	-33.5868
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0169	4.7971	0.0012	0.0048	-0.0053	0.2270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8180	5.1931	-0.0112	-0.0353	0.0878	34.7549
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8023	0.4453	0.0116	0.0373	-0.0875	-33.6904
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0086	2.8196	0.0006	0.0025	-0.0027	0.1234
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7510	4.8565	-0.0154	-0.0489	0.1191	46.2493
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0052	1.6917	0.0004	0.0015	-0.0016	0.0740
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7427	-1.4739	0.0150	0.0480	-0.1147	-45.0110
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0052	1.6917	0.0004	0.0015	-0.0016	0.0740

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.2392
Shear X	-6.2573
Shear Z	0.0262
Moment X	0.0845
Moment Y (Twist)	0.2024
Moment Z	78.1011

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	7.1707
Shear X	-3.7544
Shear Z	-0.0154
Moment X	0.0490
Moment Y (Twist)	0.1191
Moment Z	46.2987

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	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 2. D + L	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 3. D + (S or Lr or R)	0.0196	4.8542	-0.0721	-0.2622	0.0258	-0.2022
ULS: 3. D + (S or Lr or R)	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0169	4.2792	-0.0620	-0.2254	0.0221	-0.1716
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 5b. D + 0.7E	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0169	4.2792	-0.0620	-0.2254	0.0221	-0.1716
ULS: 8. 0.6D + 0.7E	0.0052	1.5326	-0.0190	-0.0691	0.0066	-0.0479
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2950	5.3047	-0.1188	-0.4017	0.5519	40.7950
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3102	-0.1952	0.0534	0.1645	-0.5180	-40.0576
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4609	6.3420	-0.1273	-0.4404	0.4277	30.4845
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0169	4.2792	-0.0620	-0.2254	0.0221	-0.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4930	2.2171	0.0018	-0.0157	-0.3747	-30.1549
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0169	4.2792	-0.0620	-0.2254	0.0221	-0.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4691	4.6171	-0.0970	-0.3300	0.4167	30.5763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4848	0.4921	0.0321	0.0946	-0.3858	-30.0631
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0086	2.5543	-0.0317	-0.1151	0.0111	-0.0798

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2985	4.2830	-0.1061	-0.3556	0.5474	40.8269
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0052	1.5326	-0.0190	-0.0691	0.0066	-0.0479
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3068	-1.2170	0.0661	0.2106	-0.5225	-40.0257
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0052	1.5326	-0.0190	-0.0691	0.0066	-0.0479

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.0373
Shear X	-5.5198
Shear Z	-0.2047
Moment X	-0.6941
Moment Y (Twist)	0.9293
Moment Z	68.7383

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.3420
Shear X	-3.3102
Shear Z	-0.1273
Moment X	-0.4404
Moment Y (Twist)	0.5519
Moment Z	40.8269

Project Details

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

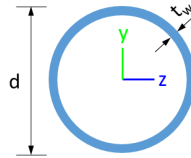


Design Input Information

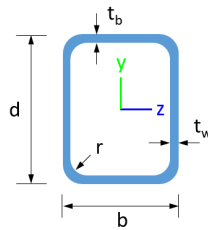
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

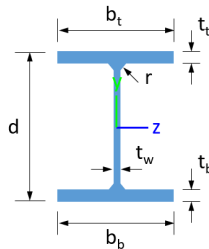
Section Dimensions



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



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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212	196.55	190.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.92	6.12	40.24	43.62
214	133.20	85.85	23.89	6.12	40.24	43.62
215	133.20	69.16	17.46	6.12	40.24	43.62
216	133.20	69.16	17.77	6.12	40.24	43.62
301	574.32	249.16	123.94	123.94	172.30	172.30
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	52.83	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	52.83	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.76	6.12	40.24	43.62
314	133.20	85.85	25.01	6.12	40.24	43.62
315	133.20	69.16	17.42	6.12	40.24	43.62
316	133.20	69.16	17.37	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.036	0.555	0.015	0.032	0.001	0.578	#13	0.534	Not Required	Pass
2	0.003	0.261	0.223	0.063	0.044	0.486	#13	0.035	Not Required	Pass
3	0.009	0.518	0.044	0.051	0.004	0.546	#13	0.045	Not Required	Pass
4	0.008	0.517	0.138	0.052	0.030	0.585	#13	0.080	Not Required	Pass
5	0.009	0.321	0.135	0.051	0.034	0.338	#13	0.074	Not Required	Pass
6	0.011	0.608	0.081	0.062	0.016	0.660	#13	0.045	Not Required	Pass
7	0.012	0.377	0.187	0.060	0.047	0.404	#13	0.074	Not Required	Pass
8	0.002	0.072	0.165	0.040	0.018	0.167	#23	0.095	Not Required	Pass
9	0.012	0.041	0.065	0.002	0.002	0.108	#13	0.204	Not Required	Pass
10	0.012	0.591	0.181	0.059	0.039	0.644	#13	0.080	Not Required	Pass
11	0.002	0.067	0.169	0.041	0.018	0.172	#23	0.095	Not Required	Pass
12	0.003	0.344	0.265	0.078	0.051	0.609	#13	0.035	Not Required	Pass
13	0.006	0.185	0.430	0.053	0.024	0.536	#21	0.286	Not Required	Pass
14	0.008	0.181	0.426	0.051	0.024	0.520	#21	0.190	Not Required	Pass
15	0.000	0.057	0.149	0.025	0.011	0.194	#21	Not Required	Not Required	Pass
16	0.000	0.057	0.149	0.025	0.011	0.194	#21	Not Required	Not Required	Pass
101	0.041	0.630	0.002	0.036	0.000	0.651	#13	0.534	Not Required	Pass
102	0.003	0.340	0.274	0.081	0.052	0.614	#13	0.035	Not Required	Pass
103	0.012	0.628	0.070	0.063	0.010	0.673	#13	0.045	Not Required	Pass
104	0.011	0.637	0.181	0.064	0.039	0.715	#13	0.080	Not Required	Pass
105	0.011	0.390	0.187	0.062	0.047	0.418	#13	0.074	Not Required	Pass
106	0.011	0.646	0.070	0.065	0.011	0.689	#13	0.045	Not Required	Pass
107	0.011	0.401	0.183	0.064	0.046	0.429	#13	0.074	Not Required	Pass
108	0.002	0.053	0.160	0.041	0.018	0.187	#21	0.095	Not Required	Pass
109	0.014	0.042	0.055	0.001	0.000	0.101	#13	0.204	Not Required	Pass
110	0.011	0.644	0.176	0.064	0.038	0.713	#13	0.080	Not Required	Pass

111	0.003	0.062	0.163	0.041	0.018	0.186	#21	0.095	Not Required	Pass
112	0.003	0.350	0.283	0.081	0.054	0.634	#13	0.035	Not Required	Pass
113	0.006	0.184	0.434	0.053	0.024	0.566	#21	0.286	Not Required	Pass
114	0.009	0.204	0.431	0.054	0.024	0.573	#21	0.286	Not Required	Pass
115	0.004	0.255	0.236	0.041	0.018	0.436	#21	0.473	Not Required	Pass
116	0.002	0.248	0.237	0.042	0.018	0.432	#21	0.473	Not Required	Pass
201	0.041	0.630	0.002	0.036	0.000	0.651	#13	0.534	Not Required	Pass
202	0.003	0.350	0.283	0.081	0.054	0.634	#13	0.035	Not Required	Pass
203	0.011	0.646	0.070	0.065	0.011	0.689	#13	0.045	Not Required	Pass
204	0.011	0.644	0.176	0.064	0.038	0.713	#13	0.080	Not Required	Pass
205	0.011	0.401	0.183	0.064	0.046	0.429	#13	0.074	Not Required	Pass
206	0.012	0.628	0.070	0.063	0.010	0.673	#13	0.045	Not Required	Pass
207	0.011	0.390	0.187	0.062	0.047	0.418	#13	0.074	Not Required	Pass
208	0.002	0.061	0.169	0.042	0.018	0.192	#21	0.095	Not Required	Pass
209	0.014	0.042	0.055	0.001	0.000	0.101	#13	0.204	Not Required	Pass
210	0.011	0.637	0.181	0.064	0.039	0.715	#13	0.080	Not Required	Pass
211	0.002	0.069	0.171	0.041	0.018	0.189	#21	0.095	Not Required	Pass
212	0.003	0.340	0.274	0.081	0.052	0.614	#13	0.035	Not Required	Pass
213	0.006	0.184	0.434	0.053	0.024	0.566	#21	0.286	Not Required	Pass
214	0.009	0.204	0.431	0.054	0.024	0.573	#21	0.286	Not Required	Pass
215	0.004	0.241	0.236	0.041	0.018	0.421	#21	0.473	Not Required	Pass
216	0.002	0.222	0.236	0.041	0.018	0.410	#21	0.473	Not Required	Pass
301	0.036	0.555	0.015	0.032	0.001	0.578	#13	0.534	Not Required	Pass
302	0.003	0.343	0.265	0.078	0.051	0.609	#13	0.035	Not Required	Pass
303	0.011	0.608	0.081	0.062	0.016	0.660	#13	0.045	Not Required	Pass
304	0.012	0.591	0.181	0.059	0.039	0.644	#13	0.080	Not Required	Pass
305	0.012	0.377	0.187	0.060	0.047	0.404	#13	0.074	Not Required	Pass
306	0.009	0.518	0.044	0.051	0.004	0.546	#13	0.045	Not Required	Pass
307	0.009	0.321	0.135	0.051	0.034	0.338	#13	0.074	Not Required	Pass
308	0.000	0.057	0.149	0.025	0.011	0.194	#21	Not Required	Not Required	Pass
309	0.012	0.041	0.065	0.002	0.002	0.108	#13	0.204	Not Required	Pass
310	0.008	0.517	0.138	0.052	0.030	0.585	#13	0.080	Not Required	Pass
311	0.000	0.057	0.149	0.025	0.011	0.194	#21	Not Required	Not Required	Pass
312	0.003	0.261	0.223	0.063	0.044	0.486	#13	0.035	Not Required	Pass
313	0.006	0.185	0.430	0.053	0.024	0.536	#21	0.190	Not Required	Pass
314	0.008	0.181	0.426	0.051	0.024	0.520	#21	0.286	Not Required	Pass
315	0.004	0.254	0.236	0.041	0.018	0.436	#21	0.473	Not Required	Pass
316	0.002	0.253	0.236	0.040	0.018	0.435	#21	0.473	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS
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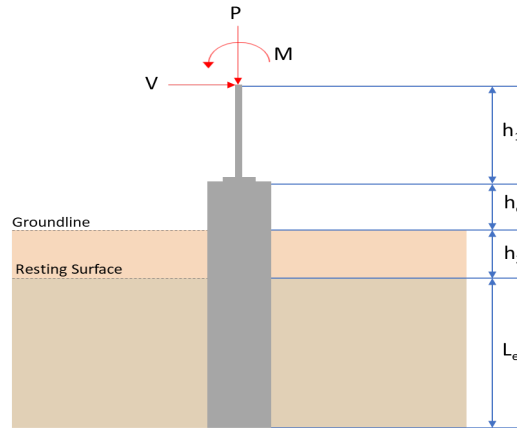
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.342	9.037
V_x (kip)	-3.310	-5.520
V_z (kip)	0.127	0.205
M_x (kipft)	0.440	0.694
M_z (kipft)	40.827	68.737

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.31 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.52707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.827 \text{ kipft}) + ((-3.31 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5011 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.7445 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.127 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.020223 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.44 \text{ kipft}) + ((0.127 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.070064 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0029 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.7445 \text{ ft}), (2.0029 \text{ ft})]$$

$$L_{e,req} = 6.745 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.745 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.93034$$

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

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

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

$$q = \frac{(6.342 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39638 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39638 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19819$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.52707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.5011 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5011 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.5011 \text{ kipft/ft})) + (4 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0034 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.5011 \text{ kipft/ft})) + (3 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.5011 \text{ kipft/ft})) + (2 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.25435 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.5011 \text{ kipft/ft})) + ((-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.048 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0034 \text{ ft})}{2}$$

$$p_a = 0.37526 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25435 \text{ kip/ft}^2)}{(0.37526 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.67781$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.048 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96368$$

Status: **PASS**
Ratio: **0.680**

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

Considering z-direction:

$H_o = 0.020223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.070064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.070064 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.020223 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.070064 \text{ kipft/ft})) + (4 \times (0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1852 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.070064 \text{ kipft/ft})) + (3 \times (0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.070064 \text{ kipft/ft})) + (2 \times (0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.014697 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.070064 \text{ kipft/ft})) + ((0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.032732 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1852 \text{ ft})}{2}$$

$$p_a = 0.38889 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.014697 \text{ kip/ft}^2)}{(0.38889 \text{ kip/ft}^2)}$$

$$Ratio = 0.037793$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.032732 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.030098$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.52 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.87898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(68.737 \text{ kipft}) + ((-5.52 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.945 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.945 \text{ kipft/ft})}{(-0.87898 \text{ kip/ft})}$$

$$E = 12.452 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.945 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (10.945 \text{ kipft/ft})) + (4 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = \frac{(-0.87898 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (10.945 \text{ kipft/ft})) + (4 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0023 \text{ ft}$$

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

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

$$V_{max} = ((-0.87898 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.452 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0023 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.452 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0023 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.005 \text{ kip}$$

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

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

$$M_{max} = ((-0.87898 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(12.452 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.452 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.452 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 44.828 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.205 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.032643 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.694 \text{ kipft}) + ((0.205 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11051 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11051 \text{ kipft/ft})}{(0.032643 \text{ kip/ft})}$$

$$E = 3.3854 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11051 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.032643 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.11051 \text{ kipft/ft})) + (4 \times (0.032643 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1886 \text{ ft}$$

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

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

$$V_{max} = ((0.032643 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1886 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1886 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19498 \text{ kip}$$

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

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

$$M_{max} = ((0.032643 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.3854 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.62243 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.037 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.296 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.296 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.037 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0033781$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.037 \text{ kip} \rightarrow 9037 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9037 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.69 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.69 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.69 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.69 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.88 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.005 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.005 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.11729$$

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

Considering z-direction:

$V_{max} = 0.19498 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.19498 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.0017584$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 44.828 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(44.828 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.1796$$

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

Considering z-direction:

$M_{max} = 0.62243 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62243 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0024937$$

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

REFERENCES	CALCULATIONS	RESULTS
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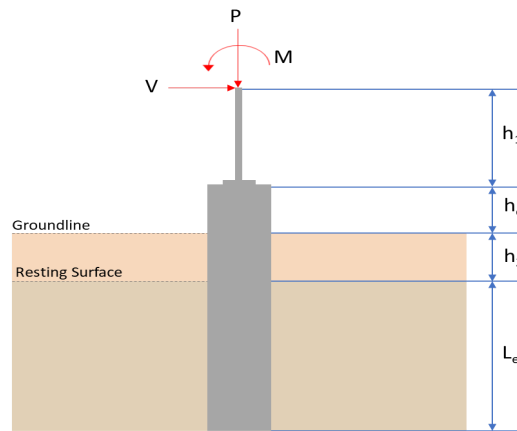
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.342	9.037
V_x (kip)	-3.310	-5.520
V_z (kip)	-0.127	-0.205
M_x (kipft)	-0.440	-0.694
M_z (kipft)	40.827	68.738

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.31 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.52707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.827 \text{ kipft}) + ((-3.31 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5011 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.7445 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.127 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.020223 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.44 \text{ kipft}) + ((-0.127 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.070064 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5501 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.7445 \text{ ft}), (1.5501 \text{ ft})]$$

$$L_{e,req} = 6.745 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(6.745 \text{ ft})}{(7.25 \text{ ft})}$$

$$Ratio = 0.93034$$

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

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

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

$$q = \frac{(6.342 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39638 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39638 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19819$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.52707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.5011 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5011 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.5011 \text{ kipft/ft})) + (4 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0034 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.5011 \text{ kipft/ft})) + (3 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (6.5011 \text{ kipft/ft})) + (2 \times (-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.25435 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.5011 \text{ kipft/ft})) + ((-0.52707 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.048 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0034 \text{ ft})}{2}$$

$$p_a = 0.37526 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25435 \text{ kip/ft}^2)}{(0.37526 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.67781$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.048 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96368$$

Status: **PASS**
Ratio: **0.680**

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

Considering z-direction:

$H_o = -0.020223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.070064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.070064 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.020223 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.070064 \text{ kipft/ft})) + (4 \times (-0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1852 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.070064 \text{ kipft/ft})) + (3 \times (-0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.070064 \text{ kipft/ft})) + (2 \times (-0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.0043765 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.070064 \text{ kipft/ft})) + ((-0.020223 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.0007407 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1852 \text{ ft})}{2}$$

$$p_a = 0.38889 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0043765 \text{ kip/ft}^2)}{(0.38889 \text{ kip/ft}^2)}$$

$$Ratio = -0.011254$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0007407 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = -0.0006811$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.52 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.87898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(68.738 \text{ kipft}) + ((-5.52 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.946 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.946 \text{ kipft/ft})}{(-0.87898 \text{ kip/ft})}$$

$$E = 12.453 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.946 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{6 \times (10.946 \text{ kipft/ft}) + 4 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft})}$$

$$a = \frac{(-0.87898 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (10.946 \text{ kip/ft})) + (4 \times (-0.87898 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0023 \text{ ft}$$

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

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

$$V_{max} = ((-0.87898 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.453 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0023 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.453 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0023 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.005 \text{ kip}$$

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

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

$$M_{max} = ((-0.87898 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(12.453 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.453 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.453 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0023 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 44.828 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.205 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.032643 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.694 \text{ kipft}) + ((-0.205 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11051 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11051 \text{ kipft/ft})}{(-0.032643 \text{ kip/ft})}$$

$$E = 3.3854 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11051 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.032643 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.11051 \text{ kipft/ft})) + (4 \times (-0.032643 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1886 \text{ ft}$$

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

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

$$V_{max} = ((-0.032643 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1886 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1886 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19498 \text{ kip}$$

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

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

$$M_{max} = ((-0.032643 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(3.3854 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3854 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1886 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.62243 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.037 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.296 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.296 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.037 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0033781$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.037 \text{ kip} \rightarrow 9037 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9037 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.69 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.69 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.69 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.69 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.88 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.005 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.005 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.11729$$

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

Considering z-direction:

$V_{max} = 0.19498 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.19498 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.0017584$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 44.828 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(44.828 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.1796$$

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

Considering z-direction:

$M_{max} = 0.62243 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.62243 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0024937$$

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

REFERENCES	CALCULATIONS	RESULTS
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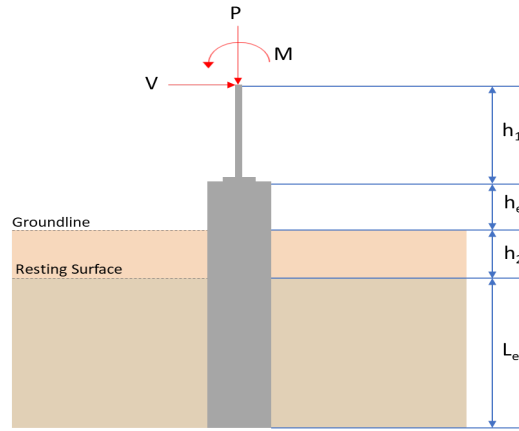
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.171	10.239
V_x (kip)	-3.754	-6.257
V_z (kip)	0.015	0.026
M_x (kipft)	-0.049	-0.084
M_z (kipft)	46.299	78.101

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.754 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.299 \text{ kipft}) + ((-3.754 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.3725 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.9766 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.015 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0023885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.049 \text{ kipft}) + ((0.015 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0078025 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.91039 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9766 \text{ ft}), (0.91039 \text{ ft})]$$

$$L_{e,req} = 6.977 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.977 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93027$$

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

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

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

$$q = \frac{(7.171 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44819 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44819 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22409$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59777 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.3725 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.3725 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.3725 \text{ kipft/ft})) + (4 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1803 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.3725 \text{ kipft/ft})) + (3 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.3725 \text{ kipft/ft})) + (2 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.26085 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.3725 \text{ kipft/ft})) + ((-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0946 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1803 \text{ ft})}{2}$$

$$p_a = 0.38852 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26085 \text{ kip/ft}^2)}{(0.38852 \text{ kip/ft}^2)}$$

$$Ratio = 0.6714$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0946 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.97295$$

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

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

Considering z-direction:

$H_o = 0.0023885 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0078025 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0078025 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.0078025 \text{ kipft/ft})) + (4 \times (0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.378 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0078025 \text{ kipft/ft})) + (3 \times (0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 [(3 \times (0.0078025 \text{ kipft/ft})) + (2 \times (0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.0016244 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0078025 \text{ kipft/ft})) + ((0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.0035754 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.378 \text{ ft})}{2}$$

$$p_a = 0.40335 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0016244 \text{ kip/ft}^2)}{(0.40335 \text{ kip/ft}^2)}$$

$$Ratio = 0.0040274$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0035754 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0031781$$

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

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



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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(78.101 \text{ kipft}) + ((-6.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.436 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.436 \text{ kipft/ft})}{(-0.99634 \text{ kip/ft})}$$

$$E = 12.482 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.436 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (12.436 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.99634 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.436 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1788 \text{ ft}$$

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

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

$$V_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1788 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1788 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.365 \text{ kip}$$

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

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

$$M_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(12.482 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.156 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0041401 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.084 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013376 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013376 \text{ kipft/ft})}{(0.0041401 \text{ kip/ft})}$$

$$E = 3.2308 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013376 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.0041401 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.013376 \text{ kipft/ft})) + (4 \times (0.0041401 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3797 \text{ ft}$$

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

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

$$V_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2308 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3797 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2308 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3797 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023682 \text{ kip}$$

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

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

$$M_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.2308 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3797 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2308 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3797 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2308 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3797 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.077752 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.256 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.256 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Minimum\ spacing\ of\ reinforcement,$</p> $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625\ in))]$ $s_{rebar} = 1.5\ in$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625\ in)), (48 \times (0.375\ in)), Min((48\ in), (48\ in))]$ $s_{ties} = 10\ in$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5\ ksi) \times [(2304\ in^2) - (4.2951\ in^2)]) + ((60\ ksi) \times (4.2951\ in^2))]$ $\phi P_N = 2675.2\ kip$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.239\ kip)}{(2675.2\ kip)}$ $Ratio = 0.0038274$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48\ in$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48\ in)$ $d = 38.4\ in$ <p>λ_s - size effect modification factor</p> $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4\ in)}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ ksi \rightarrow 2500\ psi$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ psi)} \times (48\ in) \times (38.4\ in)$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.239 \text{ kip} \rightarrow 10239 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10239 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.85 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.85 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.85 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.85 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.98 \text{ kip}$$

Considering x-direction:

V_{max} = 14.365 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.365 \text{ kip})}{(110.98 \text{ kip})}$$

$$Ratio = 0.12943$$

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

Considering z-direction:

$V_{max} = 0.023682 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.023682 \text{ kip})}{(110.98 \text{ kip})}$$

$$Ratio = 0.00021338$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 51.156 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.156 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.20495$$

Status: **PASS**
Ratio: **0.200**

Considering z-direction:

$M_{max} = 0.077752 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.077752 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00031151$$

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

REFERENCES	CALCULATIONS	RESULTS
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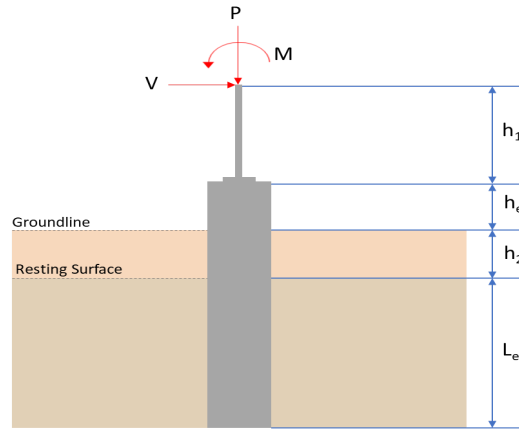
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.171	10.239
V_x (kip)	-3.754	-6.257
V_z (kip)	-0.015	0.026
M_x (kipft)	0.049	0.085
M_z (kipft)	46.299	78.101

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.754 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.59777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.299 \text{ kipft}) + ((-3.754 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.3725 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.9766 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.015 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0023885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.049 \text{ kipft}) + ((-0.015 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0078025 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.79868 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9766 \text{ ft}), (0.79868 \text{ ft})]$$

$$L_{e,req} = 6.977 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.977 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.93027$$

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

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

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

$$q = \frac{(7.171 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44819 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44819 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22409$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59777 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.3725 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.3725 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (7.3725 \text{ kipft/ft})) + (4 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1803 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.3725 \text{ kipft/ft})) + (3 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (7.3725 \text{ kipft/ft})) + (2 \times (-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.26085 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.3725 \text{ kipft/ft})) + ((-0.59777 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0946 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1803 \text{ ft})}{2}$$

$$p_a = 0.38852 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26085 \text{ kip/ft}^2)}{(0.38852 \text{ kip/ft}^2)}$$

$$Ratio = 0.6714$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0946 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.97295$$

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

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

Considering z-direction:

$H_o = -0.0023885 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0078025 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0078025 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.0078025 \text{ kipft/ft})) + (4 \times (-0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.378 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0078025 \text{ kipft/ft})) + (3 \times (-0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 [(3 \times (0.0078025 \text{ kipft/ft})) + (2 \times (-0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.000545 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0078025 \text{ kipft/ft})) + ((-0.0023885 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = -0.00024628 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.378 \text{ ft})}{2}$$

$$p_a = 0.40335 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.000545 \text{ kip/ft}^2)}{(0.40335 \text{ kip/ft}^2)}$$

$$Ratio = -0.0013512$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00024628 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = -0.00021892$$

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

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



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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(78.101 \text{ kipft}) + ((-6.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.436 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.436 \text{ kipft/ft})}{(-0.99634 \text{ kip/ft})}$$

$$E = 12.482 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.436 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (12.436 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.99634 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.436 \text{ kipft/ft})) + (4 \times (-0.99634 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1788 \text{ ft}$$

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

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

$$V_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1788 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1788 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.365 \text{ kip}$$

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

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

$$M_{max} = ((-0.99634 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(12.482 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.482 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1788 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.156 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0041401 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.085 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013535 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013535 \text{ kipft/ft})}{(0.0041401 \text{ kip/ft})}$$

$$E = 3.2692 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013535 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.0041401 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.013535 \text{ kipft/ft})) + (4 \times (0.0041401 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3779 \text{ ft}$$

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

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

$$V_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2692 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3779 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2692 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3779 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023831 \text{ kip}$$

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

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

$$M_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(3.2692 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3779 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2692 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3779 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2692 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3779 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.078306 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.256 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.256 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.239 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0038274$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.239 \text{ kip} \rightarrow 10239 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10239 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.85 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.85 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.85 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.85 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.98 \text{ kip}$$

Considering x-direction:

V_{max} = 14.365 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.365 \text{ kip})}{(110.98 \text{ kip})}$$

$$Ratio = 0.12943$$

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

Considering z-direction:

$V_{max} = 0.023831 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.023831 \text{ kip})}{(110.98 \text{ kip})}$$

$$Ratio = 0.00021472$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 51.156 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.156 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.20495$$

Status: **PASS**
Ratio: **0.200**

Considering z-direction:

$M_{max} = 0.078306 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.078306 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00031373$$

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