

Project Details



Project Name: Keystone State Park 5x11 Carport - V1Jb

Date: Thu Apr 10 2025

Location: 1150 Keystone Park Rd, Derry, PA 15627, USA

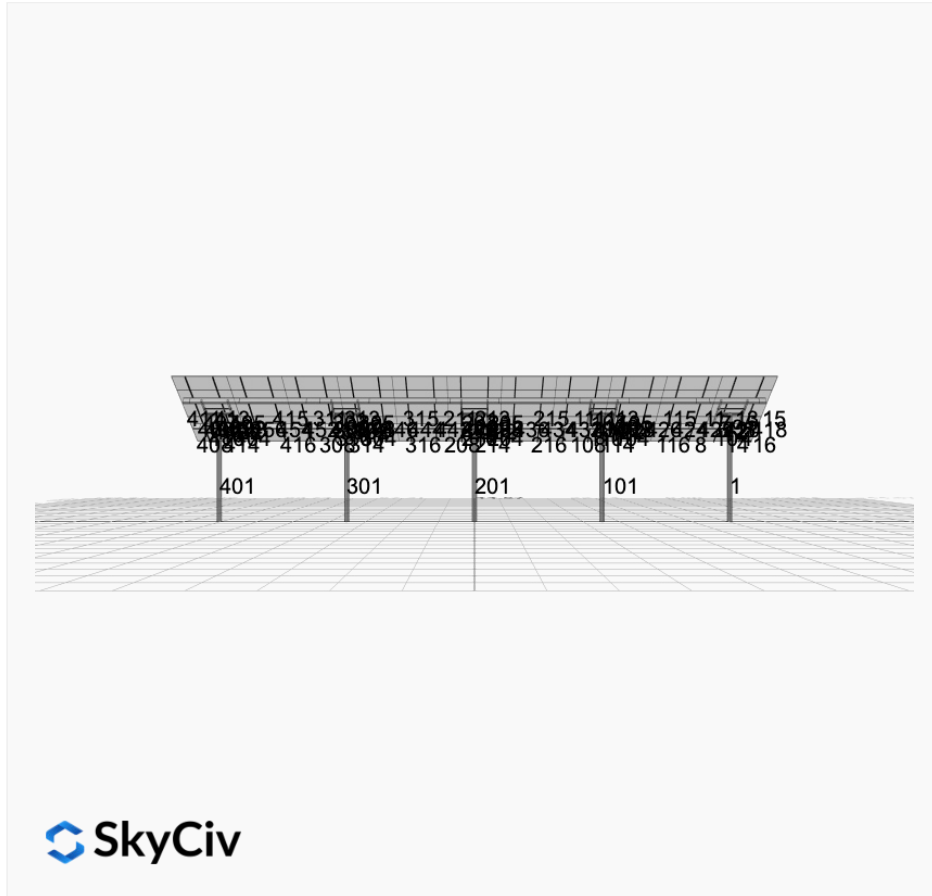
Number of Modules: 55

Number of Poles: 5

Unique ID: 5P-19.75-8TOP-XD-12-L-5Hx11W-GEFH

Date Sold:

Dealer: _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	89.74 ft
Winter Tilt Angle	30
Front Edge Clearance	12 ft

MT Solar Bill of Materials (5P-19.75-8TOP-XD-12-L-5Hx11W-GEFH)

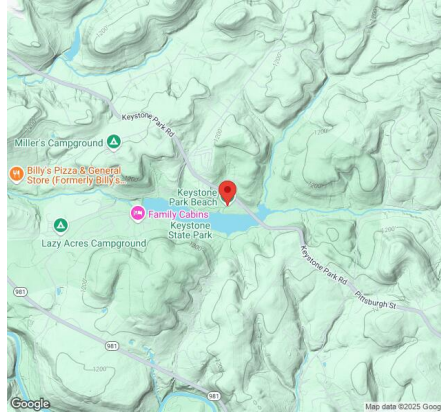
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-XD	H-Frame Assembly-XD	5
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	8
MTS-XD-Splice-57	57IN XD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	11

Rail Bill of Materials

Part	Qty
Rails (226in)	22

Part	Qty
Rail Attachment	88
Module Mid Clamp	88
Module End Clamp	44
Ground Lug	11

Site Details:



Site Address: 1150 Keystone Park Rd, Derry, PA 15627, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	96.90in
Number of Rows:	5
Number of Columns:	11
Total Number of Modules:	55
Winter Tilt Angle:	30
Front Edge Clearance:	12
Total Array Height at Tilt:	21.40 ft
Total Frame Length:	88.50 ft
Module Info/Notes:	590w
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	89.74 ft
Rail Length:	225.50 in
Rail Spacing:	4.08 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	16.70 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.25 ft Pile 2: 7.75 ft Pile 3: 7.75 ft Pile 4: 7.75 ft Pile 5: 7.25 ft
Foundation Volume:	22.370 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	1150 Keystone Park Rd, Derry, PA 15627, USA
Wind Speed:	103 mph

Snow Load:

44 psf

Design Disclaimer

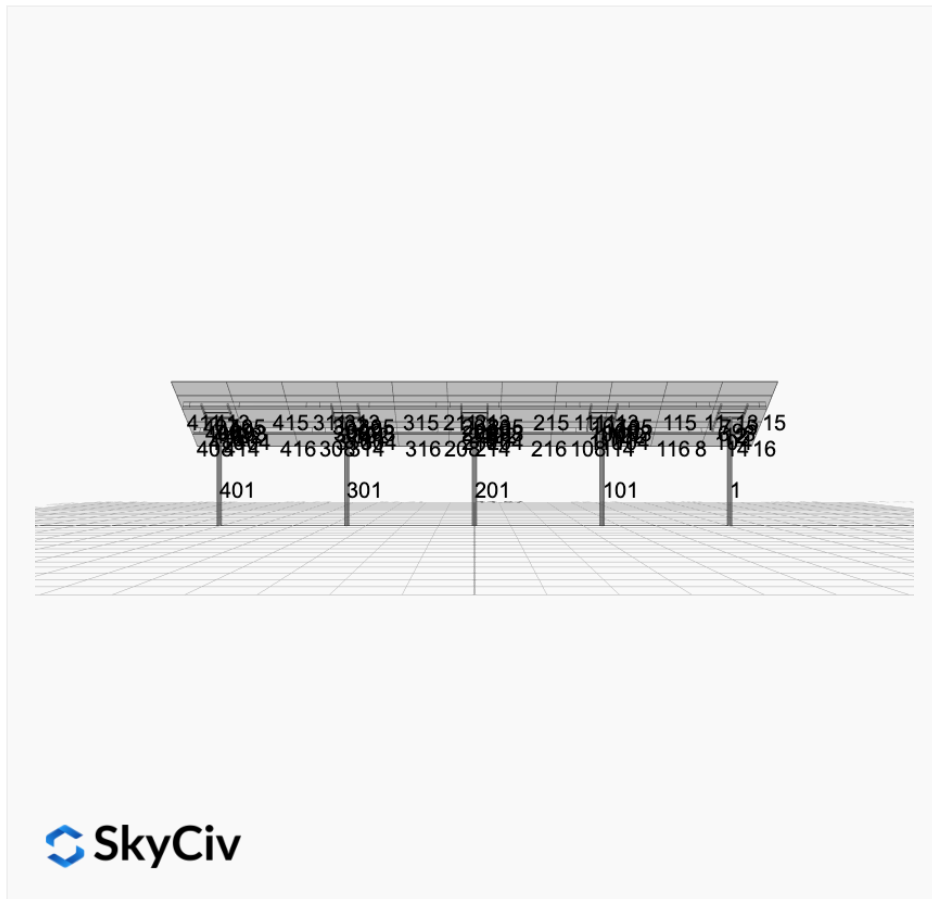
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

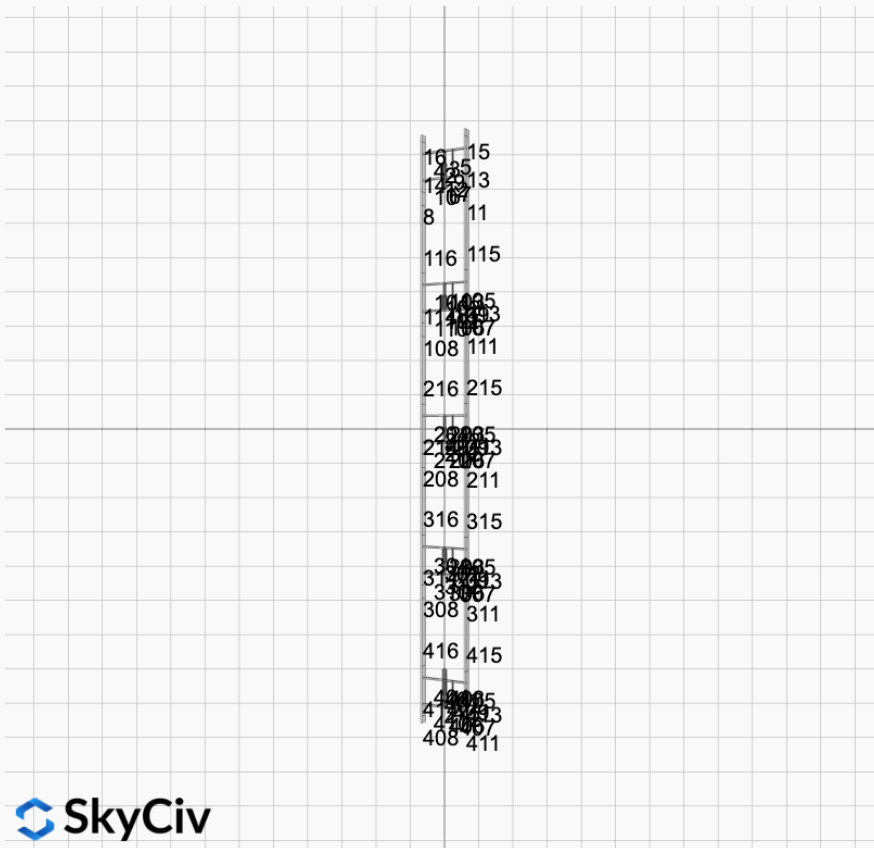
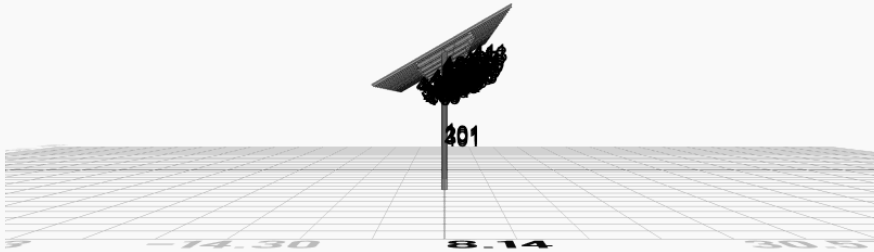
AutoDesigner Input

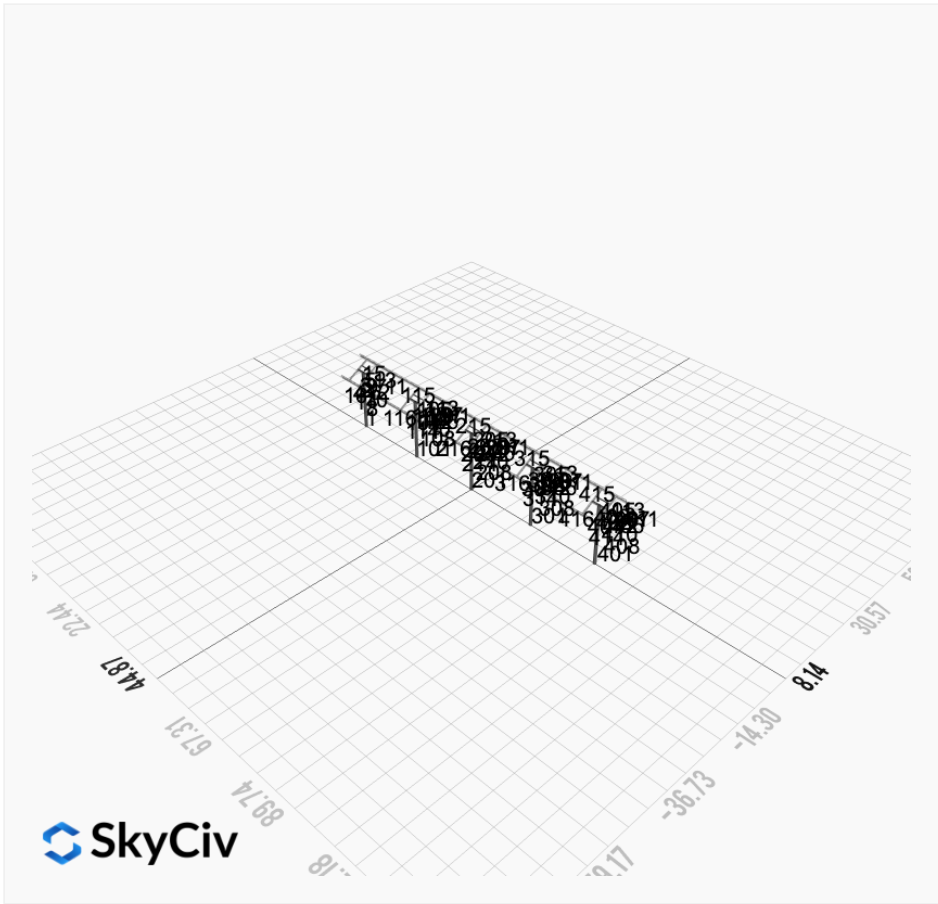
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Design Notes:

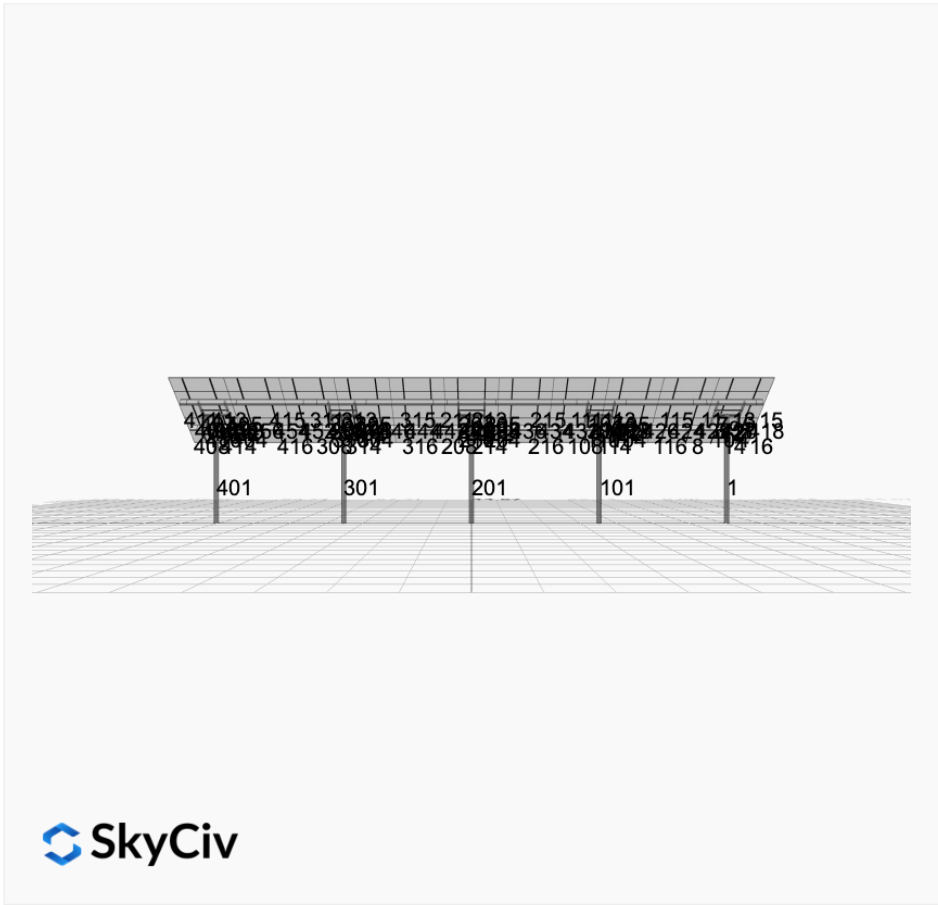
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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







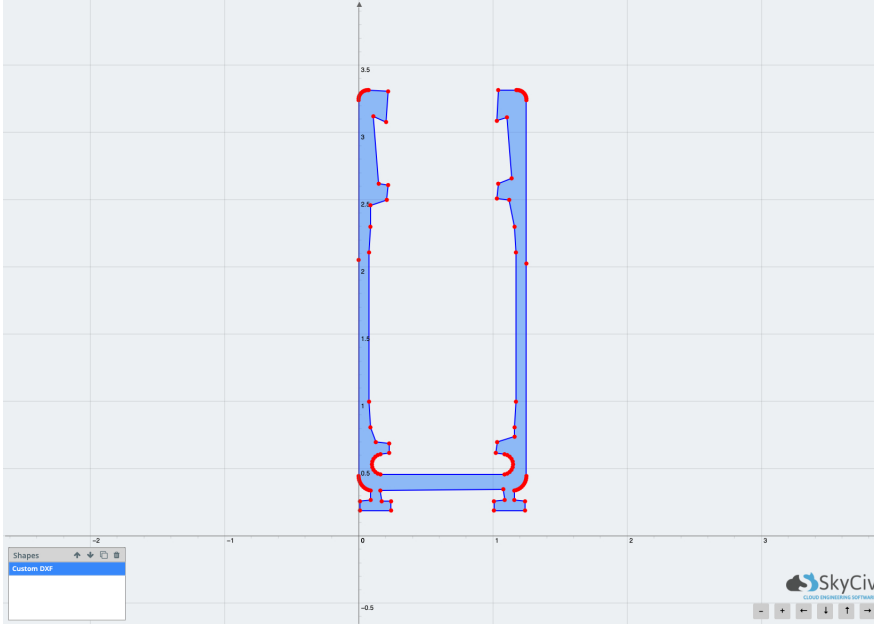
 SkyCiv



 SkyCiv

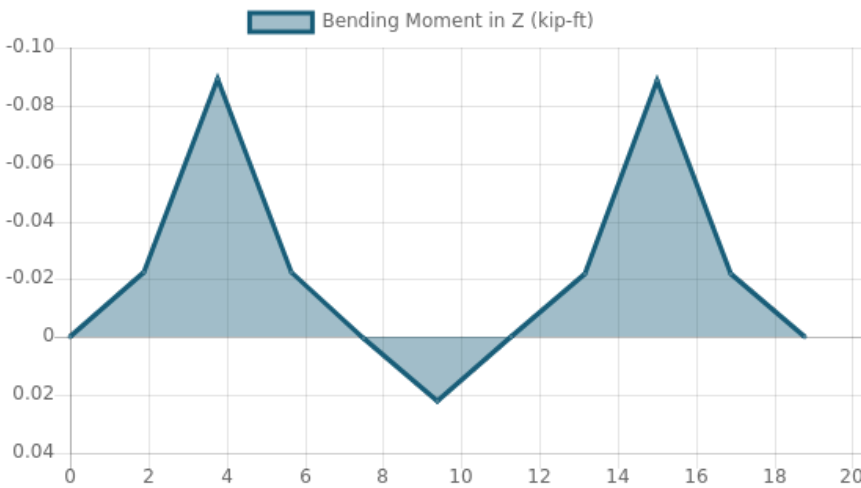
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 4.07916666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0684 kip/ft
Snow (Y): -0.0395 kip/ft
Wind uplift Case A: 0.0822 kip/ft
Wind uplift Case A: 0.0822 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1141 kip/ft

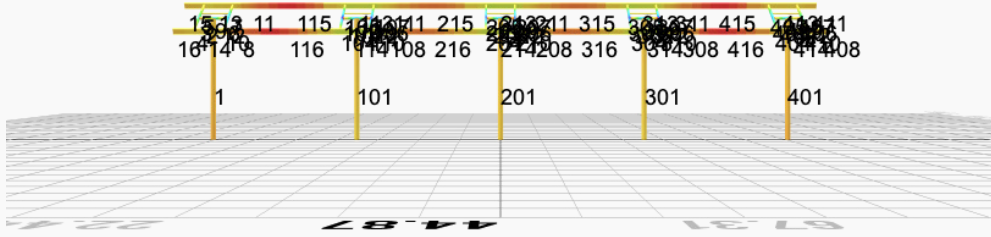
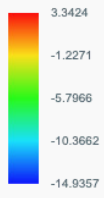


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	19.48987587	0.565	PASS
Material Yield	34.5	19.48987587	0.565	PASS
Material Strength	37	19.48987587	0.527	PASS

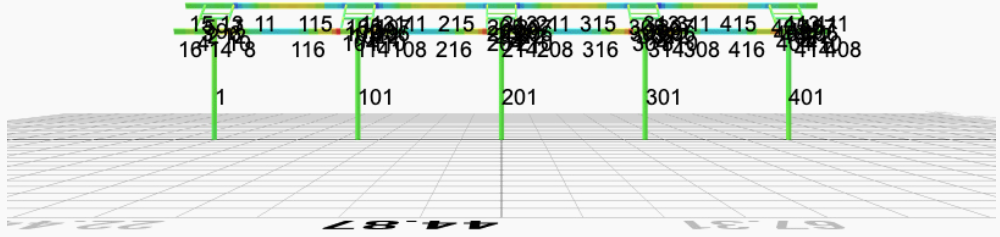
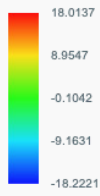
Member 1, ULS: 1. 1.4D



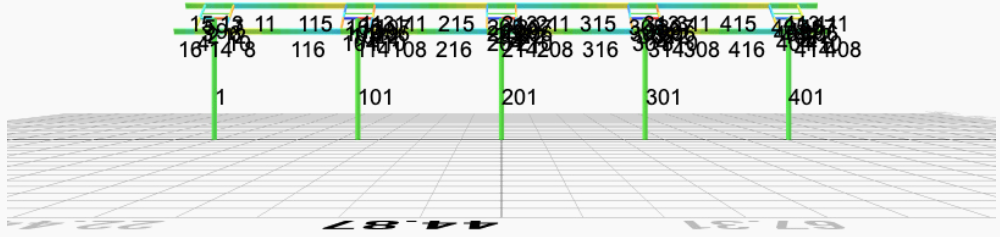
Top Bending Stress Z (ksi)



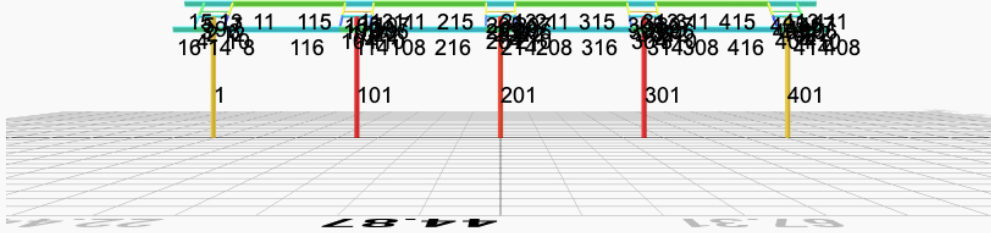
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0296	2.5059	0.0719	0.3817	-0.0684	-0.4318
ULS: 2. D + L	0.0296	2.5059	0.0719	0.3817	-0.0684	-0.4318
ULS: 3. D + (S or Lr or R)	0.1156	6.9191	0.2831	1.5059	-0.2709	-1.7714
ULS: 3. D + (S or Lr or R)	0.0296	2.5059	0.0719	0.3817	-0.0684	-0.4318
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0941	5.8158	0.2303	1.2248	-0.2203	-1.4365
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0296	2.5059	0.0719	0.3817	-0.0684	-0.4318
ULS: 5b. D + 0.7E	0.0296	2.5059	0.0719	0.3817	-0.0684	-0.4318
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0941	5.8158	0.2303	1.2248	-0.2203	-1.4365
ULS: 8. 0.6D + 0.7E	0.0177	1.5035	0.0431	0.2290	-0.0410	-0.2591
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9626	5.7814	0.3251	1.7006	-0.9040	34.0484
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.9626	5.7814	0.3251	1.7006	-0.9040	34.0484
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7259	-0.2936	-0.1358	-0.6961	0.6151	-28.4790
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4829	0.1500	-0.1283	-0.6568	0.6032	-31.2523
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4000	8.2724	0.4202	2.2140	-0.8470	24.4236
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4000	8.2724	0.4202	2.2140	-0.8470	24.4236
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3664	3.7162	0.0745	0.4164	0.2924	-22.4720
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1842	4.0489	0.0801	0.4459	0.2834	-24.5519
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4646	4.9625	0.2618	1.3709	-0.6951	25.4283
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4646	4.9625	0.2618	1.3709	-0.6951	25.4283
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3018	0.4063	-0.0839	-0.4267	0.4443	-21.4672
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1196	0.7390	-0.0783	-0.3972	0.4353	-23.5471
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9744	4.7790	0.2964	1.5479	-0.8766	34.2211
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9744	4.7790	0.2964	1.5479	-0.8766	34.2211
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7141	-1.2960	-0.1646	-0.8488	0.6425	-28.3063
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4711	-0.8524	-0.1571	-0.8095	0.6306	-31.0796

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	12.8043
Shear X	-3.3203
Shear Z	0.6449
Moment X	3.4179
Moment Y (Twist)	1.6086
Moment Z	57.9284

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	8.2724
Shear X	-1.9744
Shear Z	0.4202
Moment X	2.2140
Moment Y (Twist)	0.9040
Moment Z	34.2211

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	-0.0260	3.1869	-0.0040	-0.0222	0.0106	0.4113
ULS: 2. D + L	-0.0260	3.1869	-0.0040	-0.0222	0.0106	0.4113
ULS: 3. D + (S or Lr or R)	-0.1014	9.5902	-0.0155	-0.0861	0.0406	1.5718
ULS: 3. D + (S or Lr or R)	-0.0260	3.1869	-0.0040	-0.0222	0.0106	0.4113
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0825	7.9894	-0.0127	-0.0701	0.0331	1.2817

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0260	3.1869	-0.0040	-0.0222	0.0106	0.4113
ULS: 5b. D + 0.7E	-0.0260	3.1869	-0.0040	-0.0222	0.0106	0.4113
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0825	7.9894	-0.0127	-0.0701	0.0331	1.2817
ULS: 8. 0.6D + 0.7E	-0.0156	1.9122	-0.0024	-0.0133	0.0063	0.2468
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6457	7.8738	0.0359	0.1805	-0.1971	45.2687
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.6457	7.8738	0.0359	0.1805	-0.1971	45.2687
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2297	-0.8401	-0.0317	-0.1616	0.1623	-35.8720
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8164	-0.1379	-0.0446	-0.2285	0.2133	-38.5449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0474	11.5046	0.0173	0.0819	-0.1227	34.9247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0474	11.5046	0.0173	0.0819	-0.1227	34.9247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6092	4.9691	-0.0334	-0.1747	0.1469	-25.9308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2992	5.4958	-0.0431	-0.2248	0.1851	-27.9355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9908	6.7021	0.0260	0.1299	-0.1452	34.0544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9908	6.7021	0.0260	0.1299	-0.1452	34.0544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6658	0.1666	-0.0248	-0.1267	0.1243	-26.8012
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3558	0.6933	-0.0345	-0.1769	0.1626	-28.8058
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6353	6.5990	0.0375	0.1894	-0.2014	45.1042
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.6353	6.5990	0.0375	0.1894	-0.2014	45.1042
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2401	-2.1149	-0.0301	-0.1527	0.1580	-36.0365
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8267	-1.4126	-0.0430	-0.2196	0.2091	-38.7094

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	17.9649
Shear X	-4.4237
Shear Z	-0.0838
Moment X	-0.4310
Moment Y (Twist)	0.3884
Moment Z	78.2057

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	11.5046
Shear X	-2.6457
Shear Z	-0.0446
Moment X	-0.2285
Moment Y (Twist)	0.2133
Moment Z	45.2687

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	-0.0072	3.1312	0.0000	-0.0000	0.0000	0.1947
ULS: 2. D + L	-0.0072	3.1312	0.0000	-0.0000	0.0000	0.1947
ULS: 3. D + (S or Lr or R)	-0.0285	9.3723	0.0000	-0.0001	0.0001	0.7261
ULS: 3. D + (S or Lr or R)	-0.0072	3.1312	0.0000	-0.0000	0.0000	0.1947
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0231	7.8120	0.0000	-0.0000	0.0001	0.5932
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0072	3.1312	0.0000	-0.0000	0.0000	0.1947
ULS: 5b. D + 0.7E	-0.0072	3.1312	0.0000	-0.0000	0.0000	0.1947
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0231	7.8120	0.0000	-0.0000	0.0001	0.5932
ULS: 8. 0.6D + 0.7E	-0.0043	1.8787	0.0000	-0.0000	0.0000	0.1168
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6705	7.7957	0.0000	-0.0000	0.0000	46.3151
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.6705	7.7957	0.0000	-0.0000	0.0000	46.3151
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2779	-0.8637	0.0000	-0.0000	0.0000	-36.9572
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8923	-0.2141	-0.0000	-0.0000	0.0000	-40.1220

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	-2.0207	11.3105	0.0000	-0.0001	0.0001	35.1835
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0207	11.3105	0.0000	-0.0001	0.0001	35.1835
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6907	4.8159	0.0000	-0.0001	0.0001	-27.2707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4015	5.3031	0.0000	-0.0000	0.0001	-29.6443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0047	6.6296	0.0000	-0.0000	0.0000	34.7850
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0047	6.6296	0.0000	-0.0000	0.0000	34.7850
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7066	0.1350	0.0000	-0.0000	0.0000	-27.6692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4174	0.6222	-0.0000	-0.0000	0.0000	-30.0428
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6677	6.5433	0.0000	-0.0000	0.0000	46.2372
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.6677	6.5433	0.0000	-0.0000	0.0000	46.2372
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2808	-2.1162	0.0000	-0.0000	0.0000	-37.0351
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8952	-1.4666	-0.0000	-0.0000	0.0000	-40.1999

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	17.6381
Shear X	-4.4568
Shear Z	0.0000
Moment X	-0.0005
Moment Y (Twist)	0.0008
Moment Z	79.9610

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	11.3105
Shear X	-2.6705
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	46.3151

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	-0.0260	3.1869	0.0040	0.0222	-0.0105	0.4113
ULS: 2. D + L	-0.0260	3.1869	0.0040	0.0222	-0.0105	0.4113
ULS: 3. D + (S or Lr or R)	-0.1014	9.5902	0.0155	0.0860	-0.0403	1.5718
ULS: 3. D + (S or Lr or R)	-0.0260	3.1869	0.0040	0.0222	-0.0105	0.4113
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0825	7.9894	0.0127	0.0700	-0.0329	1.2817
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0260	3.1869	0.0040	0.0222	-0.0105	0.4113
ULS: 5b. D + 0.7E	-0.0260	3.1869	0.0040	0.0222	-0.0105	0.4113
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0825	7.9894	0.0127	0.0700	-0.0329	1.2817
ULS: 8. 0.6D + 0.7E	-0.0156	1.9122	0.0024	0.0133	-0.0063	0.2468
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6457	7.8738	-0.0359	-0.1805	0.1972	45.2687
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.6457	7.8738	-0.0359	-0.1805	0.1972	45.2687
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2297	-0.8401	0.0317	0.1616	-0.1622	-35.8720
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8164	-0.1379	0.0446	0.2285	-0.2133	-38.5449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0474	11.5046	-0.0173	-0.0820	0.1229	34.9247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0474	11.5046	-0.0173	-0.0820	0.1229	34.9247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6092	4.9691	0.0334	0.1746	-0.1467	-25.9308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2992	5.4958	0.0431	0.2247	-0.1849	-27.9355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9908	6.7021	-0.0260	-0.1299	0.1452	34.0544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9908	6.7021	-0.0260	-0.1299	0.1452	34.0544
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6658	0.1666	0.0248	0.1267	-0.1243	-26.8012
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3558	0.6933	0.0345	0.1769	-0.1626	-28.8058

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6353	6.5990	-0.0375	-0.1894	0.2014	45.1042
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.6353	6.5990	-0.0375	-0.1894	0.2014	45.1042
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2401	-2.1149	0.0301	0.1527	-0.1580	-36.0365
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8267	-1.4126	0.0430	0.2196	-0.2090	-38.7094

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	17.9650
Shear X	-4.4237
Shear Z	0.0838
Moment X	0.4315
Moment Y (Twist)	0.3883
Moment Z	78.2061

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	11.5046
Shear X	-2.6457
Shear Z	0.0446
Moment X	0.2285
Moment Y (Twist)	0.2133
Moment Z	45.2687

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0296	2.5059	-0.0719	-0.3817	0.0684	-0.4317
ULS: 2. D + L	0.0296	2.5059	-0.0719	-0.3817	0.0684	-0.4317
ULS: 3. D + (S or Lr or R)	0.1156	6.9191	-0.2831	-1.5061	0.2711	-1.7712
ULS: 3. D + (S or Lr or R)	0.0296	2.5059	-0.0719	-0.3817	0.0684	-0.4317
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0941	5.8158	-0.2303	-1.2250	0.2204	-1.4363
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0296	2.5059	-0.0719	-0.3817	0.0684	-0.4317
ULS: 5b. D + 0.7E	0.0296	2.5059	-0.0719	-0.3817	0.0684	-0.4317
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0941	5.8158	-0.2303	-1.2250	0.2204	-1.4363
ULS: 8. 0.6D + 0.7E	0.0177	1.5035	-0.0431	-0.2290	0.0410	-0.2590
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9626	5.7814	-0.3251	-1.7006	0.9040	34.0484
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.9626	5.7814	-0.3251	-1.7006	0.9040	34.0484
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7259	-0.2936	0.1358	0.6961	-0.6151	-28.4790
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4829	0.1500	0.1283	0.6568	-0.6032	-31.2522
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4000	8.2724	-0.4202	-2.2142	0.8471	24.4238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4000	8.2724	-0.4202	-2.2142	0.8471	24.4238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3663	3.7162	-0.0745	-0.4166	-0.2922	-22.4718
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1841	4.0488	-0.0802	-0.4461	-0.2833	-24.5517
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4646	4.9625	-0.2618	-1.3709	0.6951	25.4284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4646	4.9625	-0.2618	-1.3709	0.6951	25.4284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3018	0.4063	0.0839	0.4267	-0.4442	-21.4672
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1196	0.7390	0.0783	0.3972	-0.4353	-23.5471
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9744	4.7790	-0.2964	-1.5479	0.8766	34.2211
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9744	4.7790	-0.2964	-1.5479	0.8766	34.2211
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7141	-1.2960	0.1646	0.8488	-0.6425	-28.3063
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4711	-0.8524	0.1571	0.8095	-0.6306	-31.0795

Worst Case Reactions LRFD

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

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	12.8042
Shear X	-3.3203
Shear Z	-0.6450
Moment X	-3.4189
Moment Y (Twist)	1.6091
Moment Z	57.9296

Result	Value (kip, kip-ft)
Axial	8.2724
Shear X	-1.9744
Shear Z	-0.4202
Moment X	-2.2142
Moment Y (Twist)	0.9040
Moment Z	34.2211

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States

 User Name: sales@mtsolar.us
 Project Name: Keystone State Park 5x11 Carport - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
10	8in Pipe Sch 80	8.63	0.50					

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

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

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

103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.64	6.46	56.26	44.91
114	159.30	97.43	32.02	6.46	56.26	44.91
115	159.30	75.13	20.45	6.46	56.26	44.91
116	159.30	75.13	21.49	6.46	56.26	44.91
201	574.32	134.88	123.94	123.94	172.30	172.30
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.45	6.46	56.26	44.91
214	159.30	97.43	30.98	6.46	56.26	44.91
215	159.30	75.13	21.22	6.46	56.26	44.91
216	159.30	75.13	21.25	6.46	56.26	44.91
301	574.32	134.88	123.94	123.94	172.30	172.30
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	140.46	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	140.46	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	31.64	6.46	56.26	44.91
314	159.30	97.43	32.02	6.46	56.26	44.91
315	159.30	75.13	20.74	6.46	56.26	44.91
316	159.30	75.13	21.21	6.46	56.26	44.91
401	574.32	134.88	123.94	123.94	172.30	172.30
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95

407	151.65	149.10	20.17	14.14	54.12	28.95
408	159.30	137.23	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	137.23	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	97.43	35.61	6.46	56.26	44.91
414	159.30	97.43	46.90	6.46	56.26	44.91
415	159.30	75.13	20.69	6.46	56.26	44.91
416	159.30	75.13	20.42	6.46	56.26	44.91

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.095	0.467	0.059	0.019	0.004	0.533	#13	0.731	Not Required	Pass
2	0.002	0.220	0.089	0.057	0.018	0.284	#21	0.036	Not Required	Pass
3	0.005	0.391	0.024	0.037	0.006	0.404	#21	0.046	Not Required	Pass
4	0.004	0.390	0.064	0.039	0.015	0.456	#21	0.122	Not Required	Pass
5	0.005	0.243	0.029	0.039	0.008	0.248	#21	0.076	Not Required	Pass
6	0.011	0.645	0.108	0.066	0.031	0.758	#21	0.046	Not Required	Pass
7	0.012	0.399	0.184	0.063	0.045	0.435	#21	0.076	Not Required	Pass
8	0.005	0.140	0.200	0.036	0.022	0.250	#21	0.102	Not Required	Pass
9	0.006	0.078	0.086	0.004	0.005	0.149	#21	0.206	Not Required	Pass
10	0.012	0.611	0.172	0.061	0.037	0.687	#21	0.082	Not Required	Pass
11	0.008	0.133	0.208	0.039	0.022	0.243	#24	0.102	Not Required	Pass
12	0.002	0.490	0.148	0.102	0.027	0.595	#21	0.054	Not Required	Pass
13	0.011	0.071	0.514	0.051	0.028	0.521	#24	0.306	Not Required	Pass
14	0.005	0.068	0.503	0.048	0.028	0.521	#24	0.204	Not Required	Pass
15	0.000	0.004	0.012	0.007	0.004	0.016	#21	Not Required	Not Required	Pass
16	0.000	0.004	0.012	0.007	0.004	0.016	#21	Not Required	Not Required	Pass
101	0.133	0.631	0.008	0.026	0.000	0.689	#13	0.731	Not Required	Pass
102	0.004	0.511	0.163	0.114	0.026	0.642	#21	0.054	Not Required	Pass
103	0.011	0.724	0.069	0.072	0.009	0.798	#21	0.046	Not Required	Pass
104	0.011	0.746	0.203	0.074	0.044	0.884	#21	0.082	Not Required	Pass
105	0.011	0.449	0.210	0.071	0.054	0.502	#21	0.076	Not Required	Pass
106	0.011	0.741	0.066	0.074	0.010	0.802	#21	0.046	Not Required	Pass
107	0.011	0.460	0.187	0.073	0.048	0.512	#21	0.076	Not Required	Pass
108	0.006	0.049	0.196	0.042	0.022	0.225	#24	0.102	Not Required	Pass
109	0.019	0.066	0.051	0.002	0.001	0.116	#21	0.206	Not Required	Pass
110	0.011	0.742	0.181	0.074	0.040	0.861	#21	0.082	Not Required	Pass
111	0.008	0.069	0.204	0.041	0.022	0.230	#24	0.102	Not Required	Pass
112	0.004	0.518	0.176	0.113	0.032	0.654	#21	0.036	Not Required	Pass
113	0.012	0.186	0.571	0.057	0.029	0.713	#21	0.306	Not Required	Pass
114	0.011	0.226	0.566	0.060	0.029	0.750	#21	0.306	Not Required	Pass
115	0.014	0.407	0.275	0.045	0.023	0.688	#21	0.507	Not Required	Pass
116	0.006	0.387	0.272	0.048	0.023	0.660	#21	0.507	Not Required	Pass
201	0.131	0.645	0.000	0.026	0.000	0.700	#13	0.731	Not Required	Pass
202	0.003	0.506	0.170	0.111	0.030	0.637	#21	0.036	Not Required	Pass
203	0.011	0.731	0.065	0.072	0.010	0.800	#21	0.046	Not Required	Pass
204	0.010	0.717	0.176	0.071	0.038	0.839	#21	0.082	Not Required	Pass
205	0.011	0.454	0.183	0.072	0.047	0.501	#21	0.076	Not Required	Pass

205	0.011	0.454	0.163	0.072	0.047	0.501	#21	0.076	Not Required	Pass
206	0.011	0.731	0.065	0.072	0.010	0.800	#21	0.046	Not Required	Pass
207	0.011	0.454	0.183	0.072	0.047	0.501	#21	0.076	Not Required	Pass
208	0.006	0.048	0.190	0.042	0.022	0.229	#21	0.102	Not Required	Pass
209	0.017	0.059	0.047	0.001	0.000	0.114	#21	0.206	Not Required	Pass
210	0.010	0.717	0.176	0.071	0.038	0.839	#21	0.082	Not Required	Pass
211	0.008	0.060	0.195	0.043	0.022	0.230	#21	0.102	Not Required	Pass
212	0.003	0.506	0.170	0.111	0.030	0.637	#21	0.036	Not Required	Pass
213	0.012	0.224	0.497	0.055	0.028	0.701	#21	0.306	Not Required	Pass
214	0.012	0.234	0.491	0.054	0.028	0.698	#21	0.306	Not Required	Pass
215	0.015	0.272	0.274	0.043	0.022	0.550	#21	0.507	Not Required	Pass
216	0.010	0.245	0.271	0.042	0.022	0.516	#21	0.507	Not Required	Pass
301	0.133	0.631	0.008	0.026	0.000	0.689	#13	0.731	Not Required	Pass
302	0.004	0.518	0.176	0.113	0.032	0.654	#21	0.036	Not Required	Pass
303	0.011	0.741	0.066	0.074	0.010	0.802	#21	0.046	Not Required	Pass
304	0.011	0.742	0.181	0.074	0.040	0.861	#21	0.082	Not Required	Pass
305	0.011	0.460	0.187	0.073	0.048	0.512	#21	0.076	Not Required	Pass
306	0.011	0.724	0.069	0.072	0.009	0.798	#21	0.046	Not Required	Pass
307	0.011	0.449	0.210	0.071	0.054	0.502	#21	0.076	Not Required	Pass
308	0.005	0.084	0.248	0.048	0.023	0.265	#21	0.102	Not Required	Pass
309	0.019	0.066	0.051	0.002	0.001	0.116	#21	0.206	Not Required	Pass
310	0.011	0.746	0.203	0.074	0.044	0.884	#21	0.082	Not Required	Pass
311	0.008	0.104	0.253	0.045	0.023	0.266	#21	0.102	Not Required	Pass
312	0.004	0.511	0.163	0.114	0.026	0.642	#21	0.054	Not Required	Pass
313	0.012	0.186	0.571	0.057	0.029	0.714	#21	0.306	Not Required	Pass
314	0.011	0.226	0.566	0.060	0.029	0.750	#21	0.306	Not Required	Pass
315	0.015	0.276	0.275	0.041	0.022	0.554	#21	0.507	Not Required	Pass
316	0.010	0.246	0.271	0.042	0.022	0.517	#21	0.507	Not Required	Pass
401	0.095	0.467	0.059	0.019	0.004	0.533	#13	0.731	Not Required	Pass
402	0.002	0.490	0.148	0.102	0.027	0.595	#21	0.054	Not Required	Pass
403	0.011	0.645	0.108	0.066	0.031	0.758	#21	0.046	Not Required	Pass
404	0.012	0.611	0.172	0.061	0.037	0.687	#21	0.082	Not Required	Pass
405	0.012	0.399	0.184	0.063	0.045	0.435	#21	0.076	Not Required	Pass
406	0.005	0.391	0.024	0.037	0.006	0.404	#21	0.046	Not Required	Pass
407	0.005	0.243	0.029	0.039	0.008	0.248	#21	0.076	Not Required	Pass
408	0.000	0.004	0.012	0.007	0.004	0.016	#21	Not Required	Not Required	Pass
409	0.006	0.078	0.086	0.004	0.005	0.149	#21	0.206	Not Required	Pass
410	0.004	0.390	0.064	0.039	0.015	0.456	#21	0.122	Not Required	Pass
411	0.000	0.004	0.012	0.007	0.004	0.016	#21	Not Required	Not Required	Pass
412	0.002	0.220	0.089	0.057	0.018	0.284	#21	0.036	Not Required	Pass
413	0.011	0.071	0.514	0.051	0.028	0.521	#24	0.204	Not Required	Pass
414	0.005	0.068	0.504	0.048	0.028	0.522	#24	0.306	Not Required	Pass
415	0.014	0.421	0.275	0.039	0.022	0.697	#21	0.507	Not Required	Pass
416	0.006	0.415	0.268	0.036	0.022	0.684	#21	0.507	Not Required	Pass

Definitions

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

F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/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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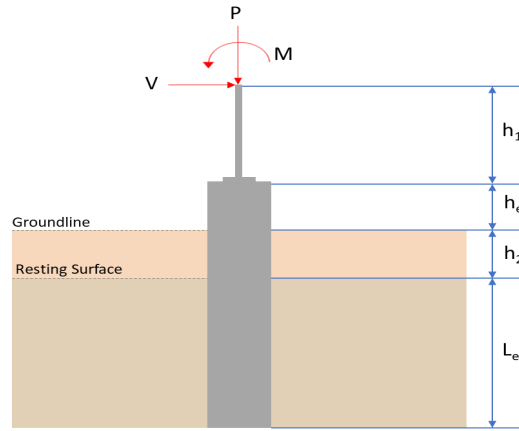
SkyCiv Foundation Design

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)	8.272	12.804
V_x (kip)	-1.974	-3.320
V_z (kip)	0.420	0.645
M_x (kipft)	2.214	3.418
M_z (kipft)	34.221	57.928

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.974 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.31433 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.221 \text{ kipft}) + ((-1.974 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.4492 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.757 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.42 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.066879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.214 \text{ kipft}) + ((0.42 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.35255 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.4808 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.757 \text{ ft}), (3.4808 \text{ ft})]$$

$$L_{e,req} = 6.757 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.757 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.932$$

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

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

$$q = \frac{(8.272 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.517 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.517 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2585$$

Status: **PASS**
Ratio: **0.260**

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:

$H_o = -0.31433 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.4492 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.4492 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.4492 \text{ kipft/ft})) + (4 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9651 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.4492 \text{ kipft/ft})) + (3 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.4492 \text{ kipft/ft})) + (2 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.27086 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.4492 \text{ kipft/ft})) + ((-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.98392 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9651 \text{ ft})}{2}$$

$$p_a = 0.37238 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27086 \text{ kip/ft}^2)}{(0.37238 \text{ kip/ft}^2)}$$

$$Ratio = 0.72738$$

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

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

$$Ratio = \frac{(0.98392 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.90475$$

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

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.066879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.35255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.35255 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.066879 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.35255 \text{ kipft/ft})) + (4 \times (0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1223 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.35255 \text{ kipft/ft})) + (3 \times (0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 [(3 \times (0.35255 \text{ kipft/ft})) + (2 \times (0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.057762 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.35255 \text{ kipft/ft})) + ((0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.13583 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1223 \text{ ft})}{2}$$

$$p_a = 0.38417 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.057762 \text{ kip/ft}^2)}{(0.38417 \text{ kip/ft}^2)}$$

$$Ratio = 0.15035$$

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

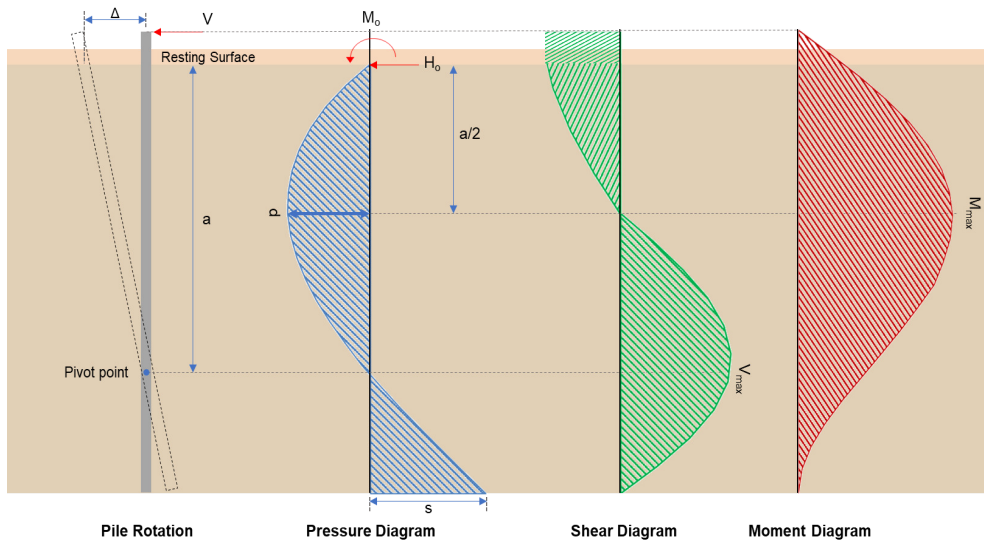
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.13583 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.12491$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.52866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.928 \text{ kipft}) + ((-3.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.2242 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.2242 \text{ kipft/ft})}{(-0.52866 \text{ kip/ft})}$$

$$E = 17.448 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.2242 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (9.2242 \text{ kipft/ft})) + (4 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = \frac{(-0.52866 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (9.2242 \text{ kipft/ft})) + (4 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9644 \text{ ft}$$

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

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

$$V_{max} = ((-0.52866 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.448 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9644 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.448 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9644 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.405 \text{ kip}$$

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

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

$$M_{max} = ((-0.52866 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(17.448 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.448 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.448 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 36.319 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.645 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10271 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(3.418 \text{ kipft}) + ((0.645 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.54427 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.54427 \text{ kipft/ft})}{(0.10271 \text{ kip/ft})}$$

$$E = 5.2992 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.54427 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.10271 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.54427 \text{ kipft/ft})) + (4 \times (0.10271 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1215 \text{ ft}$$

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

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

$$V_{max} = ((0.10271 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2992 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1215 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.2992 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1215 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.80361 \text{ kip}$$

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

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

$$M_{max} = ((0.10271 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(5.2992 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.2992 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2992 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.646 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.804 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.171 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.171 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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;">$Ratio = \frac{(12.804 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0047862$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.804 \text{ kip} \rightarrow 12804 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12804 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.21 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.405 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.405 \text{ kip})}{(111.21 \text{ kip})}$$

$$Ratio = 0.093562$$

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

Considering z-direction:

$V_{max} = 0.80361 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.80361 \text{ kip})}{(111.21 \text{ kip})}$$

$$Ratio = 0.0072263$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 36.319 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(36.319 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.14551$$

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

Considering z-direction:

$M_{max} = 2.646 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.646 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010601$$

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

REFERENCES	CALCULATIONS	RESULTS
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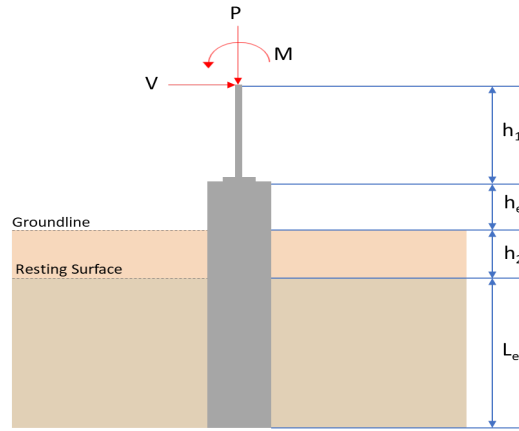
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.25$ 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)	8.272	12.804
V_x (kip)	-1.974	-3.320
V_z (kip)	-0.420	-0.645
M_x (kipft)	-2.214	-3.419
M_z (kipft)	34.221	57.930

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.974 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.31433 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.221 \text{ kipft}) + ((-1.974 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.4492 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.757 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.42 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.066879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.214 \text{ kipft}) + ((-0.42 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.35255 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.608 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.757 \text{ ft}), (2.608 \text{ ft})]$$

$$L_{e,req} = 6.757 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.757 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.932$$

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

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

$$q = \frac{(8.272 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.517 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.517 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2585$$

Status: **PASS**
Ratio: **0.260**

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:

$H_o = -0.31433 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.4492 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.4492 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.4492 \text{ kipft/ft})) + (4 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9651 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.4492 \text{ kipft/ft})) + (3 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.4492 \text{ kipft/ft})) + (2 \times (-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.27086 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.4492 \text{ kipft/ft})) + ((-0.31433 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.98392 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9651 \text{ ft})}{2}$$

$$p_a = 0.37238 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27086 \text{ kip/ft}^2)}{(0.37238 \text{ kip/ft}^2)}$$

$$Ratio = 0.72738$$

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

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

$$Ratio = \frac{(0.98392 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.90475$$

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

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.066879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.35255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.35255 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.066879 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.35255 \text{ kipft/ft})) + (4 \times (-0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1223 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.35255 \text{ kipft/ft})) + (3 \times (-0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.35255 \text{ kipft/ft})) + (2 \times (-0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.0003204 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.35255 \text{ kipft/ft})) + ((-0.066879 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.025138 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1223 \text{ ft})}{2}$$

$$p_a = 0.38417 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0003204 \text{ kip/ft}^2)}{(0.38417 \text{ kip/ft}^2)}$$

$$Ratio = 0.000834$$

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

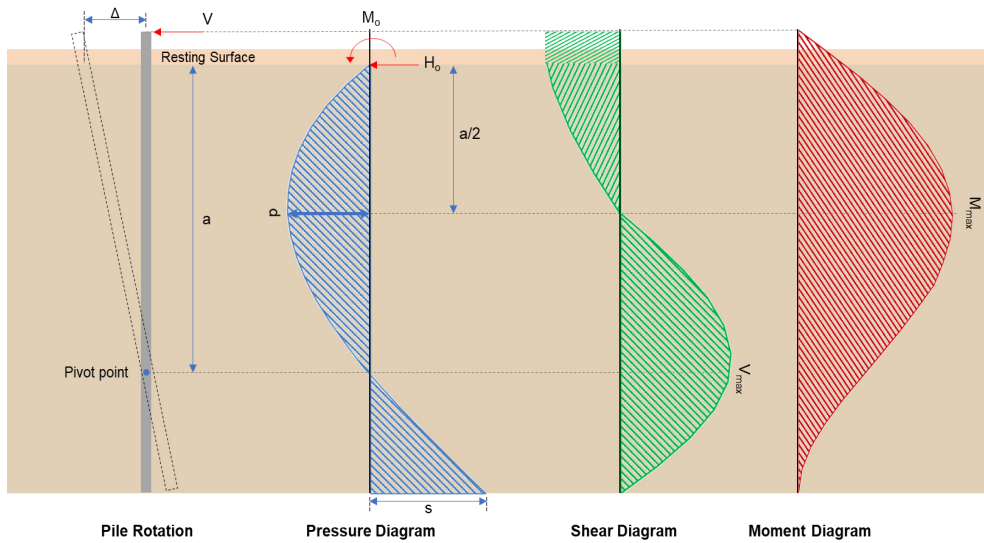
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.025138 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.023116$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.52866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.93 \text{ kipft}) + ((-3.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.2245 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.2245 \text{ kipft/ft})}{(-0.52866 \text{ kip/ft})}$$

$$E = 17.449 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.2245 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (9.2245 \text{ kipft/ft})) + (4 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = \frac{(-0.52866 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (9.2245 \text{ kipft/ft})) + (4 \times (-0.52866 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9644 \text{ ft}$$

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

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

$$V_{max} = ((-0.52866 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.449 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9644 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.449 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9644 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.405 \text{ kip}$$

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

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

$$M_{max} = ((-0.52866 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(17.449 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.449 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.449 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9644 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 36.32 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.645 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10271 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(3.419 \text{ kipft}) + ((-0.645 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.54443 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.54443 \text{ kipft/ft})}{(-0.10271 \text{ kip/ft})}$$

$$E = 5.3008 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.54443 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.10271 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.54443 \text{ kipft/ft})) + (4 \times (-0.10271 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1215 \text{ ft}$$

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

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

$$V_{max} = ((-0.10271 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.3008 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1215 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.3008 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1215 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.80377 \text{ kip}$$

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

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

$$M_{max} = ((-0.10271 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(5.3008 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.3008 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.3008 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1215 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.6466 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.804 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.171 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.171 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 - Capacity</i></p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(12.804 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0047862$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.804 \text{ kip} \rightarrow 12804 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12804 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.21 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.405 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.405 \text{ kip})}{(111.21 \text{ kip})}$$

$$Ratio = 0.093565$$

Considering z-direction:

$V_{max} = 0.80377 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.80377 \text{ kip})}{(111.21 \text{ kip})}$$

$$Ratio = 0.0072277$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 36.32 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(36.32 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.14551$$

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

Considering z-direction:

$M_{max} = 2.6466 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.6466 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010603$$

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

REFERENCES	CALCULATIONS	RESULTS
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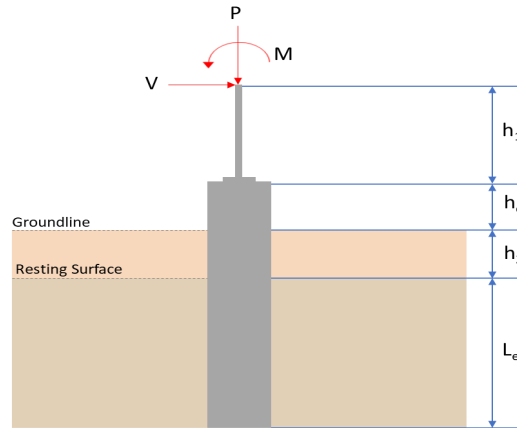
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 7.75$ 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)	11.505	17.965
V_x (kip)	-2.646	-4.424
V_z (kip)	-0.045	-0.084
M_x (kipft)	-0.228	-0.431
M_z (kipft)	45.269	78.206

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,

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

$$H_o = \frac{(-2.646 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.269 \text{ kipft}) + ((-2.646 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2084 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3168 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.045 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0071656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.228 \text{ kipft}) + ((-0.045 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036306 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3265 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3168 \text{ ft}), (1.3265 \text{ ft})]$$

$$L_{e,req} = 7.317 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.317 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.94413$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(11.505 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71906 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71906 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35953$$

Status: **PASS**
Ratio: **0.360**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42134 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.2084 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2084 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (7.2084 \text{ kipft/ft})) + (4 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3165 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.2084 \text{ kipft/ft})) + (3 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (7.2084 \text{ kipft/ft})) + (2 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.29982 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.2084 \text{ kipft/ft})) + ((-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.114 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3165 \text{ ft})}{2}$$

$$p_a = 0.39873 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29982 \text{ kip/ft}^2)}{(0.39873 \text{ kip/ft}^2)}$$

$$Ratio = 0.75193$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.114 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.95827$$

Status: **PASS**
Ratio: **0.750**

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

Considering z-direction:

$H_o = -0.0071656 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.036306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.036306 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.036306 \text{ kipft/ft})) + (4 \times (-0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4927 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.036306 \text{ kipft/ft})) + (3 \times (-0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.036306 \text{ kipft/ft})) + (2 \times (-0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = -0.0026546 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.036306 \text{ kipft/ft})) + ((-0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 0.001706 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4927 \text{ ft})}{2}$$

$$p_a = 0.41196 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0026546 \text{ kip/ft}^2)}{(0.41196 \text{ kip/ft}^2)}$$

$$Ratio = -0.0064438$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

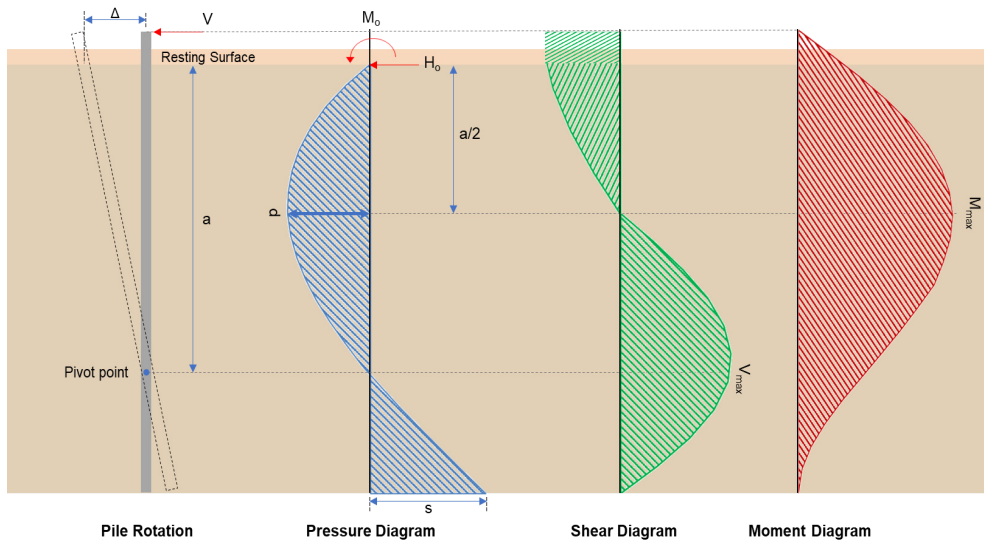
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.001706 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0014675$$

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,

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

$$H_o = \frac{(-4.424 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70446 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(78.206 \text{ kipft}) + ((-4.424 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.453 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.453 \text{ kipft/ft})}{(-0.70446 \text{ kip/ft})}$$

$$E = 17.678 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.453 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (12.453 \text{ kipft/ft})) + (4 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = \frac{(-0.70446 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.453 \text{ kipft/ft})) + (4 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3127 \text{ ft}$$

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

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

$$V_{max} = ((-0.70446 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3127 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3127 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.237 \text{ kip}$$

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

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

$$M_{max} = ((-0.70446 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(17.678 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 49.302 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.084 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.013376 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.431 \text{ kipft}) + ((-0.084 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.068631 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.068631 \text{ kipft/ft})}{(-0.013376 \text{ kip/ft})}$$

$$E = 5.131 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.068631 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.013376 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.068631 \text{ kipft/ft})) + (4 \times (-0.013376 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4907 \text{ ft}$$

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

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

$$V_{max} = ((-0.013376 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.131 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.4907 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.131 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.4907 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.098183 \text{ kip}$$

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

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

$$M_{max} = ((-0.013376 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(5.131 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.4907 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.131 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.4907 \text{ ft})}{(2 \times (7.75 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (5.131 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.4907 \text{ ft})}{(2 \times (7.75 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 0.34333 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.965 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(17.965 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0067154$	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.965 \text{ kip} \rightarrow 17965 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(17965 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.88 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.88 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.88 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.88 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.65 \text{ kip}$$

Considering x-direction:

V_{max} = 13.237 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.237 \text{ kip})}{(111.65 \text{ kip})}$$

$$Ratio = 0.11855$$

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

Considering z-direction:

$V_{max} = 0.098183 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.098183 \text{ kip})}{(111.65 \text{ kip})}$$

$$Ratio = 0.00087935$$

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:

$M_{max} = 49.302 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(49.302 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19752$$

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

Considering z-direction:

$M_{max} = 0.34333 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34333 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0013755$$

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

REFERENCES	CALCULATIONS	RESULTS
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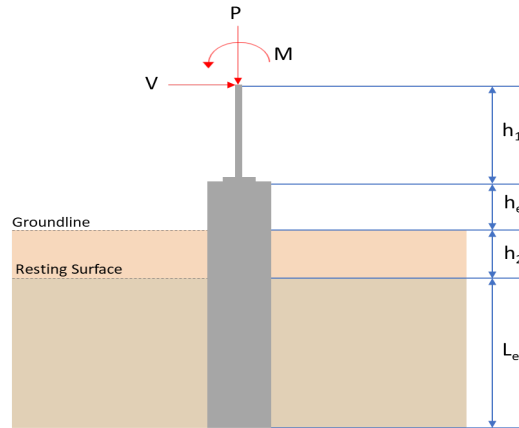
SkyCiv Foundation Design

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

$L = 7.75$ 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)	11.310	17.638
V_x (kip)	-2.671	-4.457
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	46.315	79.961

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,

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

$$H_o = \frac{(-2.671 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42532 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.315 \text{ kipft}) + ((-2.671 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.375 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3785 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3785 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.379 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.379 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.95213$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(11.31 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.70688 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.70688 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35344$$

Status: **PASS**
Ratio: **0.350**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42532 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.375 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.375 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.42532 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (7.375 \text{ kipft/ft})) + (4 \times (-0.42532 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3149 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.375 \text{ kipft/ft})) + (3 \times (-0.42532 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (7.375 \text{ kipft/ft})) + (2 \times (-0.42532 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.30919 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.375 \text{ kipft/ft})) + ((-0.42532 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.1442 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3149 \text{ ft})}{2}$$

$$p_a = 0.39862 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30919 \text{ kip/ft}^2)}{(0.39862 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77566$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

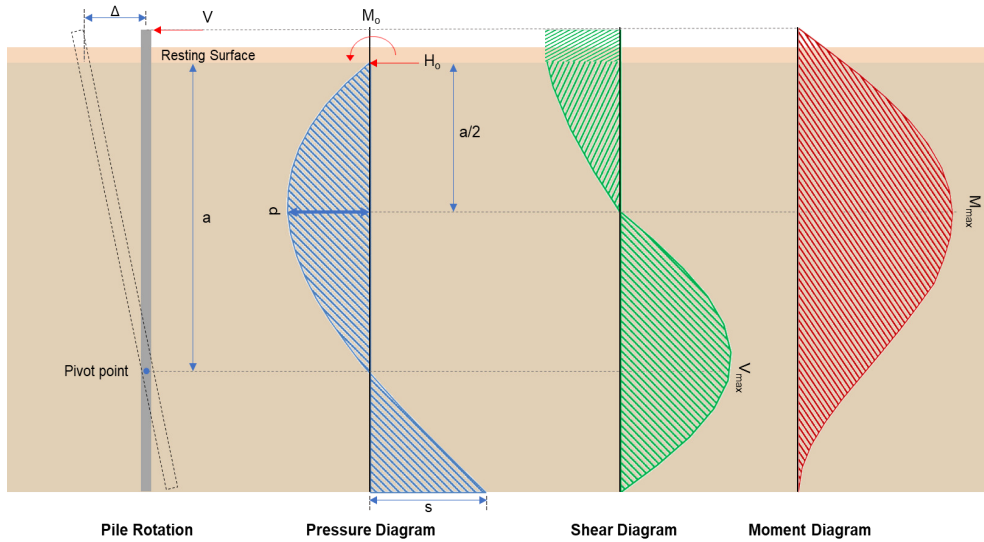
$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1442 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.780**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.457 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70971 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(79.961 \text{ kipft}) + ((-4.457 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.733 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.733 \text{ kipft/ft})}{(-0.70971 \text{ kip/ft})}$$

$$E = 17.941 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.733 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.70971 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (12.733 \text{ kipft/ft})) + (4 \times (-0.70971 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3111 \text{ ft}$$

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

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

$$V_{max} = ((-0.70971 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.941 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3111 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.941 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3111 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.500 \text{ kip}$$

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

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

$$M_{max} = ((-0.70971 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(17.941 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3111 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.941 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3111 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (17.941 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3111 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 50.331 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{17.638 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.01 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.01 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(17.638 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0065932$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.638 \text{ kip} \rightarrow 17638 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(17638 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.84 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.84 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.84 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.84 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.63 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.506 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.506 \text{ kip})}{(111.63 \text{ kip})}$$

$$\text{Ratio} = 0.12099$$

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

Flexural Strength (ACI 318-19, LFRD) 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: $M_{max} = 50.331 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(50.331 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.20165$$

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

REFERENCES	CALCULATIONS	RESULTS
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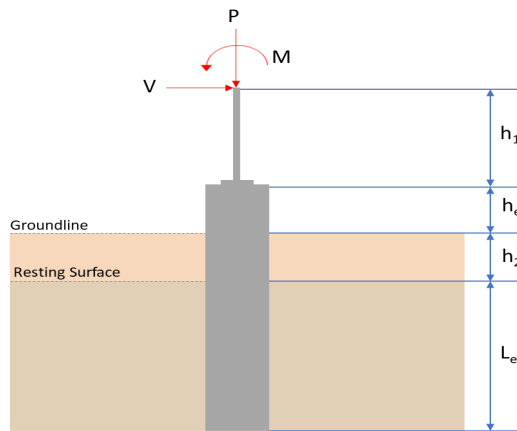
SkyCiv Foundation Design

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

$L = 7.75$ 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)	11.505	17.965
V_x (kip)	-2.646	-4.424
V_z (kip)	0.045	0.084
M_x (kipft)	0.228	0.432
M_z (kipft)	45.269	78.206

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,

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

$$H_o = \frac{(-2.646 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.269 \text{ kipft}) + ((-2.646 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2084 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.3168 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.045 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0071656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.228 \text{ kipft}) + ((0.045 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036306 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5272 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3168 \text{ ft}), (1.5272 \text{ ft})]$$

$$L_{e,req} = 7.317 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.317 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.94413$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(11.505 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71906 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71906 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35953$$

Status: **PASS**
Ratio: **0.360**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42134 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 7.2084 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2084 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (7.2084 \text{ kipft/ft})) + (4 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3165 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.2084 \text{ kipft/ft})) + (3 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (7.2084 \text{ kipft/ft})) + (2 \times (-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.29982 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (7.2084 \text{ kipft/ft})) + ((-0.42134 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.114 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3165 \text{ ft})}{2}$$

$$p_a = 0.39873 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29982 \text{ kip/ft}^2)}{(0.39873 \text{ kip/ft}^2)}$$

$$Ratio = 0.75193$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.114 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.95827$$

Status: **PASS**
Ratio: **0.750**

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

Considering z-direction:

$H_o = 0.0071656 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.036306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.036306 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.036306 \text{ kipft/ft})) + (4 \times (0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4927 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.036306 \text{ kipft/ft})) + (3 \times (0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.036306 \text{ kipft/ft})) + (2 \times (0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.0055193 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.036306 \text{ kipft/ft})) + ((0.0071656 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 0.012801 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4927 \text{ ft})}{2}$$

$$p_a = 0.41196 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0055193 \text{ kip/ft}^2)}{(0.41196 \text{ kip/ft}^2)}$$

$$Ratio = 0.013398$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

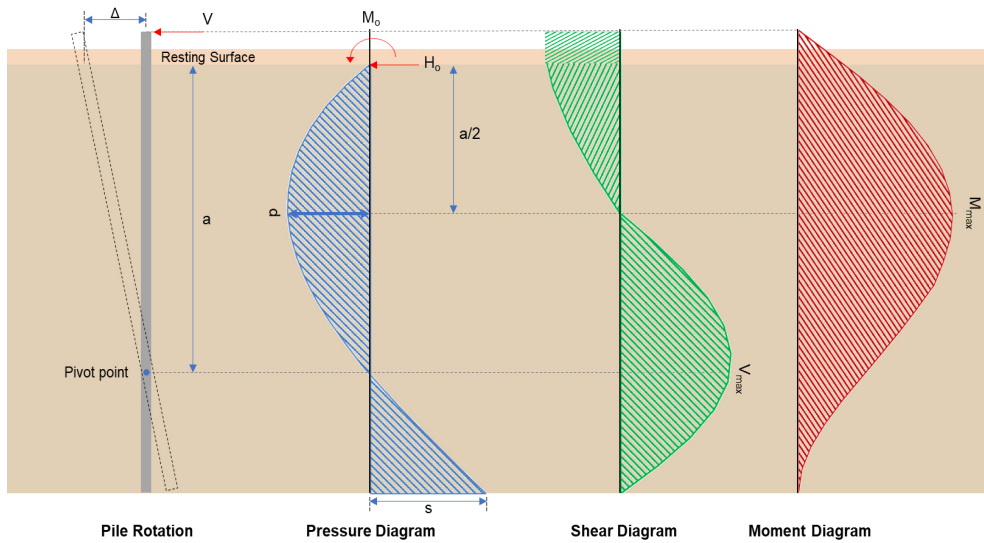
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.012801 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = 0.011012$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.424 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70446 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(78.206 \text{ kipft}) + ((-4.424 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.453 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.453 \text{ kipft/ft})}{(-0.70446 \text{ kip/ft})}$$

$$E = 17.678 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.453 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{6 \times (12.453 \text{ kipft/ft}) + 4 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft})}$$

$$a = \frac{(-0.70446 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.453 \text{ kipft/ft})) + (4 \times (-0.70446 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3127 \text{ ft}$$

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

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

$$V_{max} = ((-0.70446 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3127 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3127 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.237 \text{ kip}$$

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

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

$$M_{max} = ((-0.70446 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(17.678 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.678 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3127 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 49.302 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.084 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.013376 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.432 \text{ kipft}) + ((0.084 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.06879 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.06879 \text{ kipft/ft})}{(0.013376 \text{ kip/ft})}$$

$$E = 5.1429 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.06879 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.013376 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.06879 \text{ kipft/ft})) + (4 \times (0.013376 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4903 \text{ ft}$$

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

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

$$V_{max} = ((0.013376 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1429 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.4903 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.1429 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.4903 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.098327 \text{ kip}$$

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

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

$$M_{max} = ((0.013376 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(5.1429 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.4903 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.1429 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.4903 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1429 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.4903 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.34388 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.965 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(17.965 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0067154$	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.965 \text{ kip} \rightarrow 17965 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(17965 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.88 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.88 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.88 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.88 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.65 \text{ kip}$$

Considering x-direction:

V_{max} = 13.237 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.237 \text{ kip})}{(111.65 \text{ kip})}$$

$$Ratio = 0.11855$$

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

Considering z-direction:

$V_{max} = 0.098327 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.098327 \text{ kip})}{(111.65 \text{ kip})}$$

$$Ratio = 0.00088064$$

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:

$M_{max} = 49.302 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(49.302 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.19752$$

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

Considering z-direction:

$M_{max} = 0.34388 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34388 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0013777$$

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