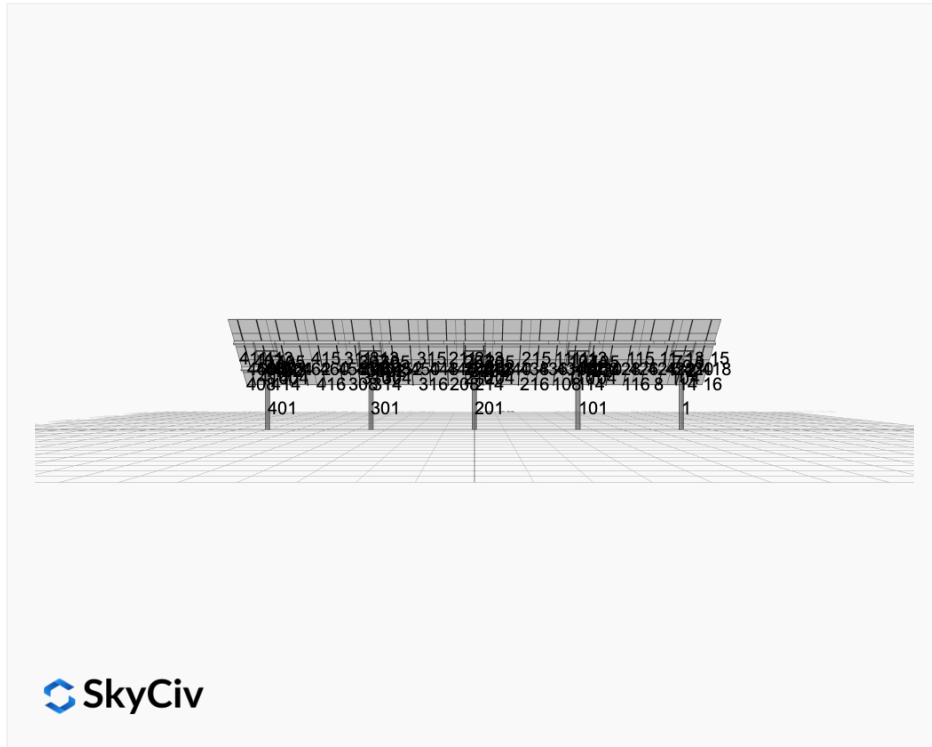


Project Details



Project Name: BT SIL530 40d 60inSquare - V1Jb **Date:** Wed Apr 30 2025
Location: 31842 Frontage Rd, Bozeman, MT 59715, USA **Number of Modules:** 65
Unique ID: 5P-19.75-10TOP-XD-24-L-5Hx13W-7I54 **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	90.57 ft
Winter Tilt Angle	40
Front Edge Clearance	8 ft

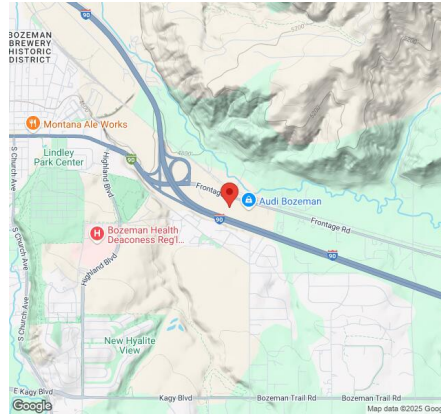
MT Solar Bill of Materials (5P-19.75-10TOP-XD-24-L-5Hx13W-7I54)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	5
MTS-HF-XD	H-Frame Assembly-XD	5
MTS-XD-Wing-24	24IN 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	13

Rail Bill of Materials

Part	Qty
Rails (226in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: 31842 Frontage Rd, Bozeman, MT 59715, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	40
Front Edge Clearance:	8
Total Array Height at Tilt:	20.08 ft
Total Frame Length:	90.50 ft
Module Info/Notes:	sil-530-xm
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	90.57 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	10in Pipe Sch 80
Pole Length above Grade:	14.04 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: 8.50 ft Pile 2: 9.00 ft Pile 3: 9.00 ft Pile 4: 9.00 ft Pile 5: 8.50 ft
Foundation Volume:	26.074 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	31842 Frontage Rd, Bozeman, MT 59715, USA
Wind Speed:	115 mph

Snow Load:

51 psf

Design Disclaimer

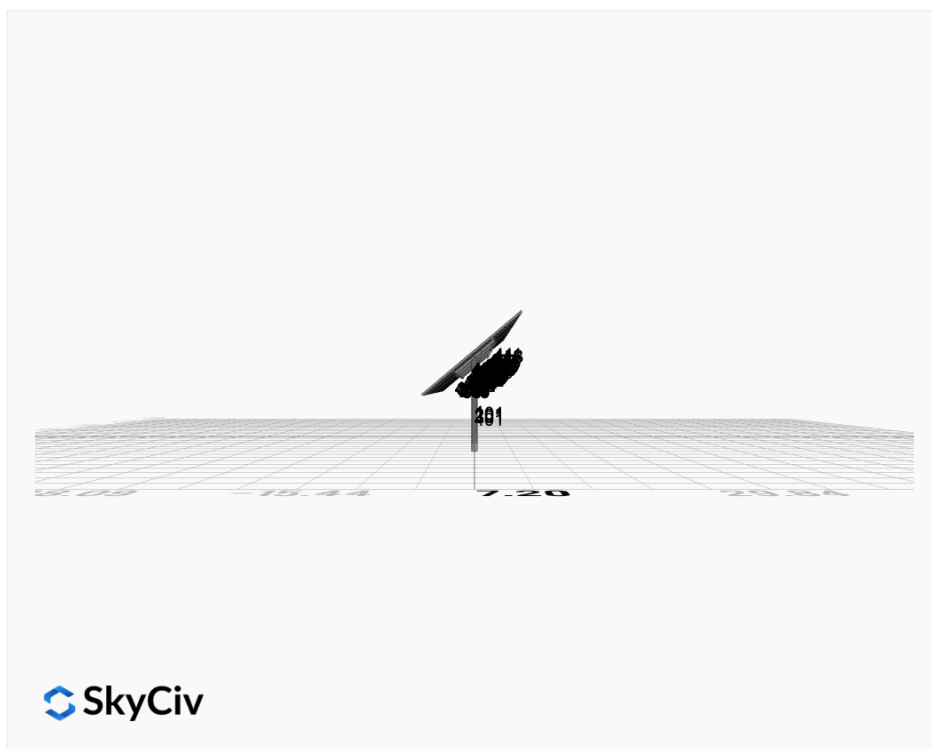
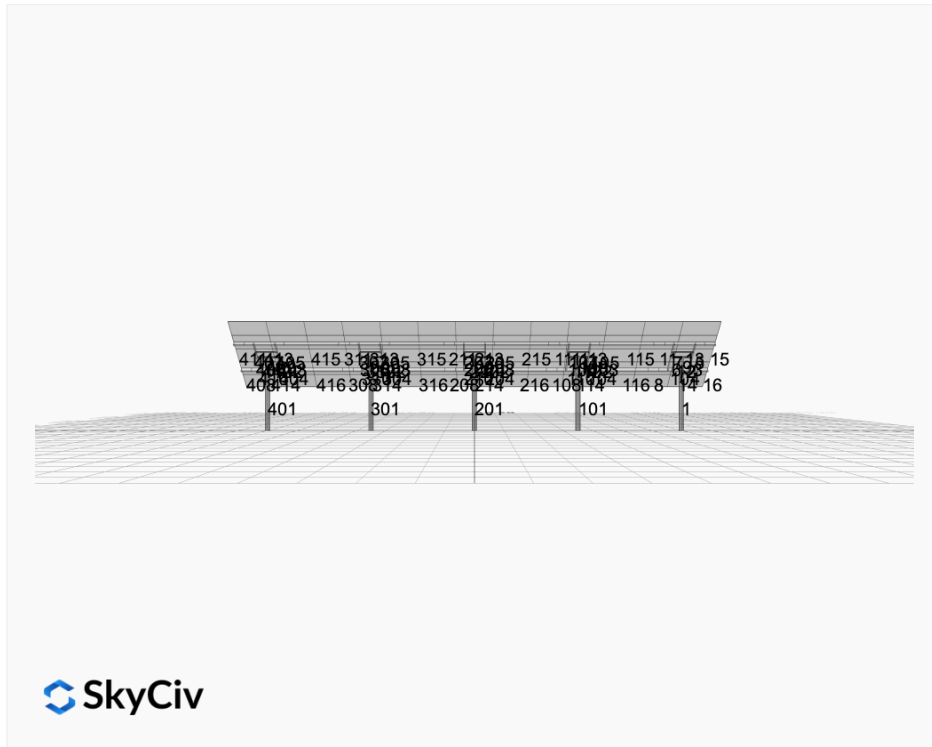
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

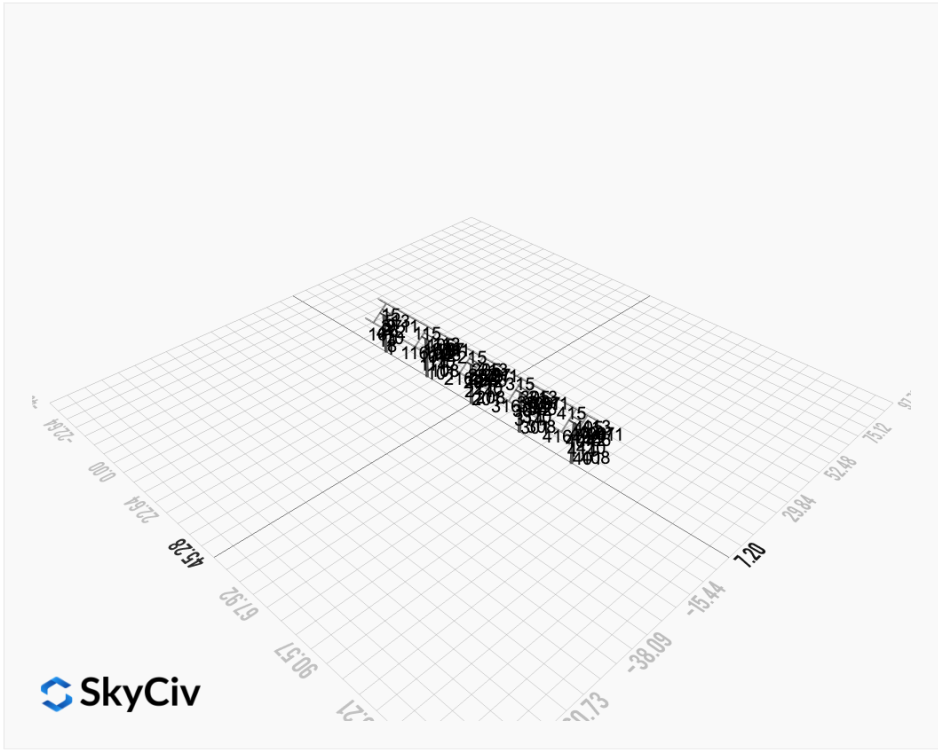
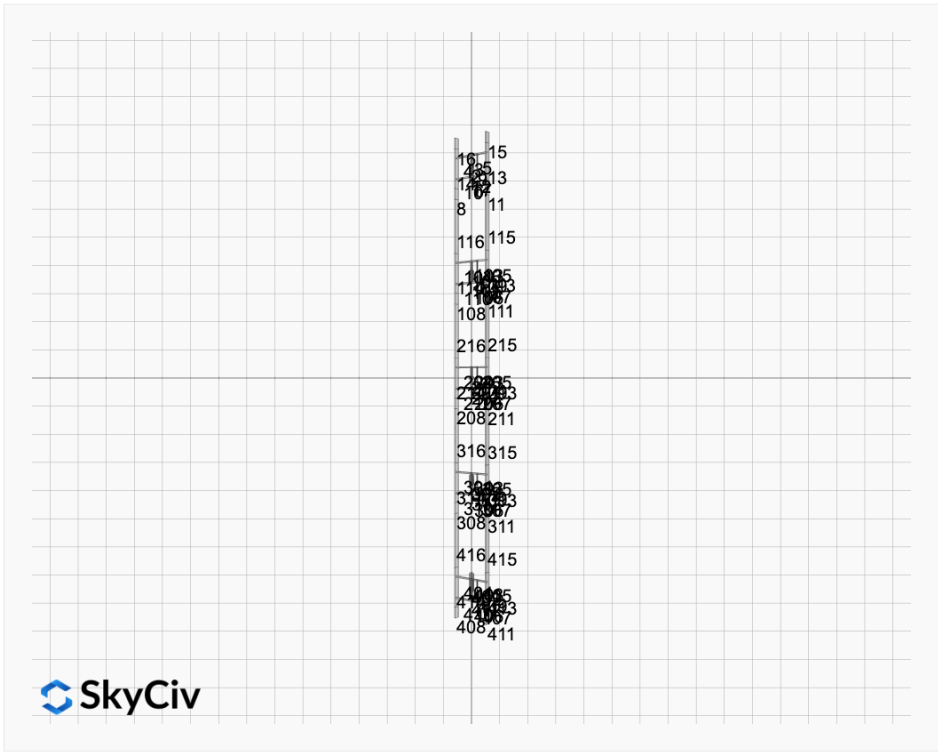
AutoDesigner Input

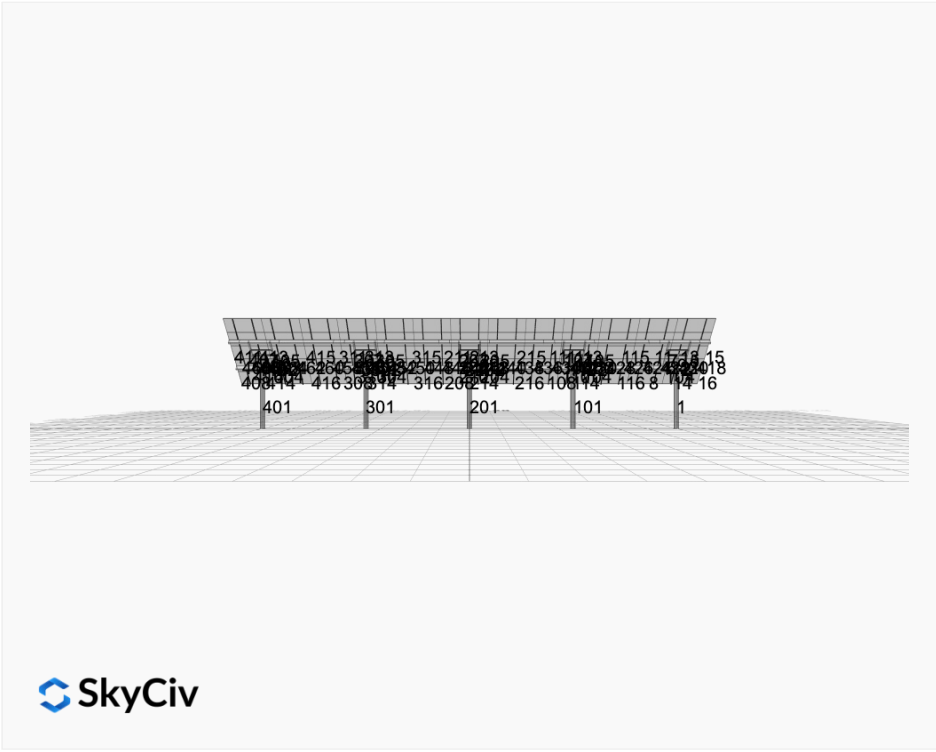
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Design Notes:

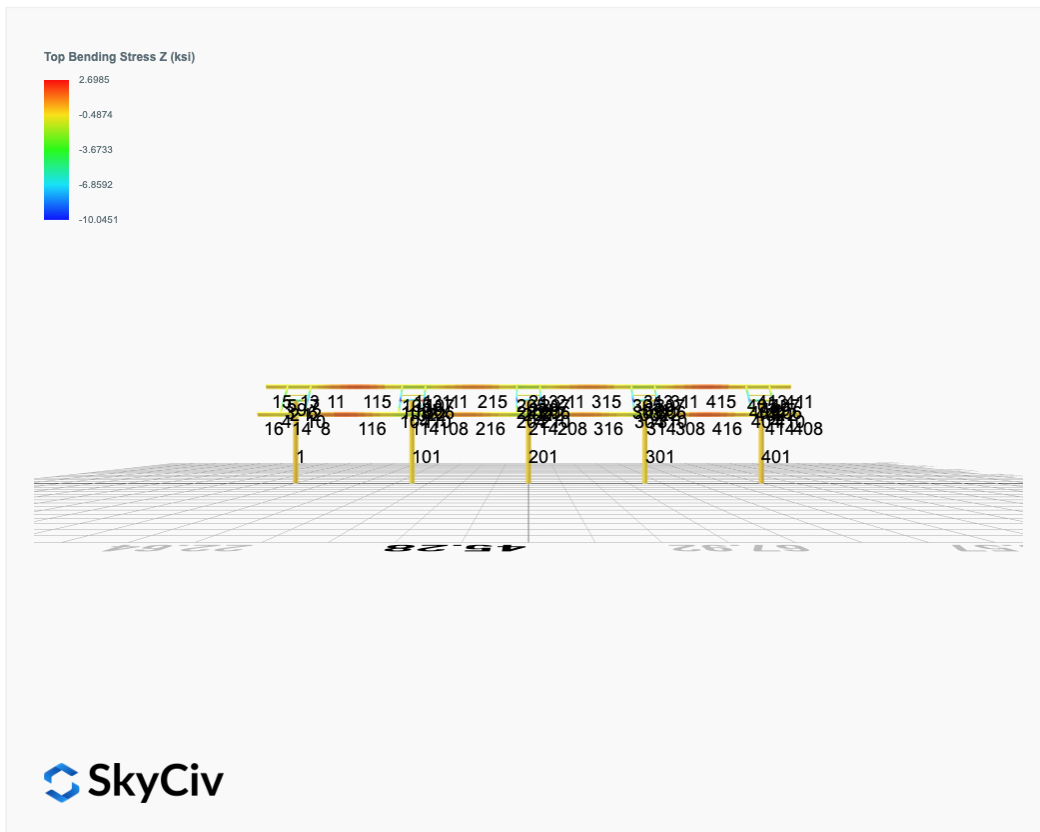
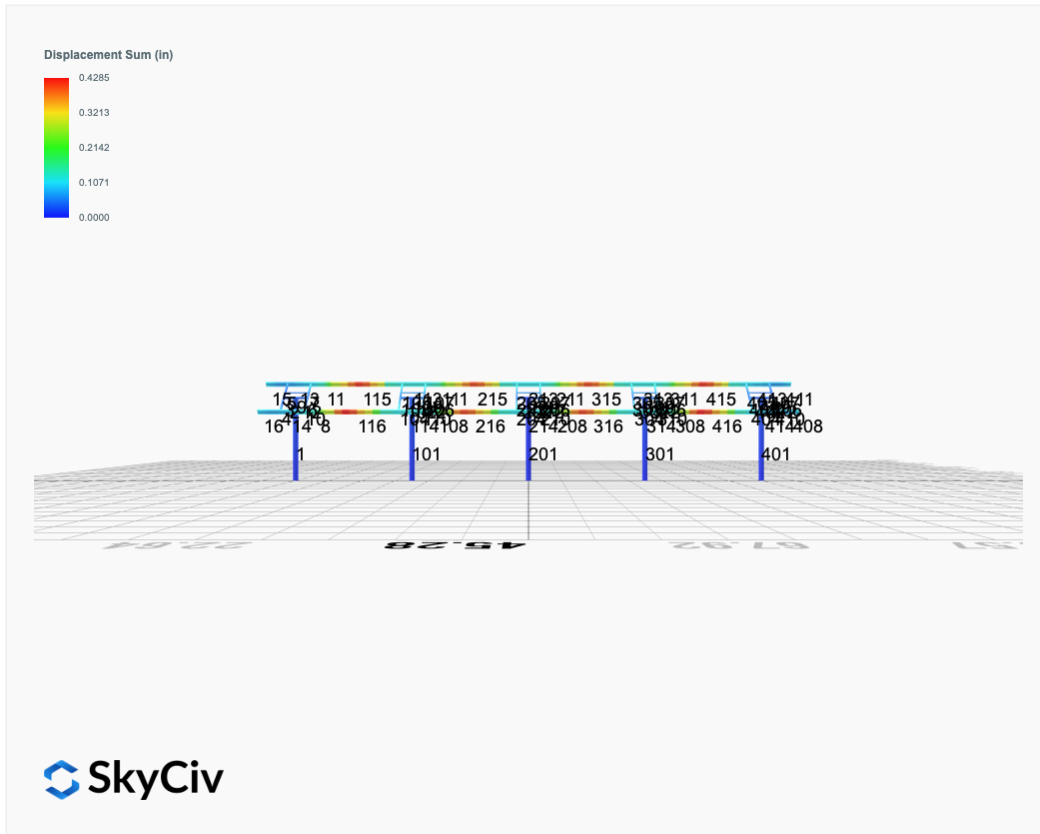
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



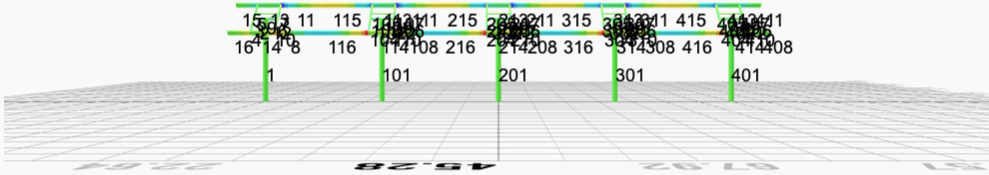
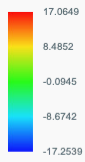




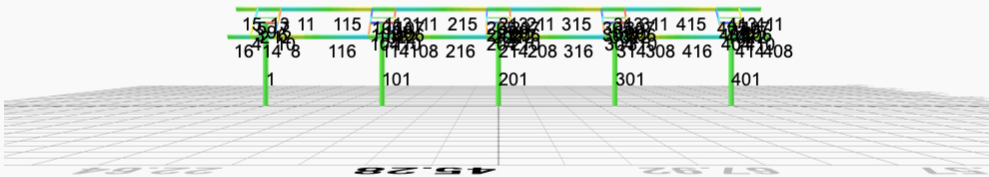
FEM Results (Envelope Worst Case for each member)

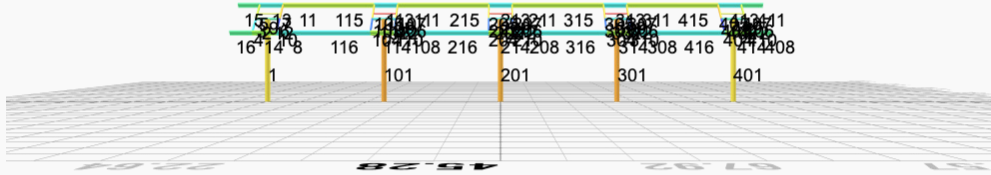
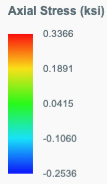


Top Bending Stress Y (ksi)



Shear Stress Y (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	0.0299	2.8223	0.0659	0.2549	-0.0557	-0.3807
ULS: 2. D + L	0.0299	2.8223	0.0659	0.2549	-0.0557	-0.3807
ULS: 3. D + (S or Lr or R)	0.0973	6.5192	0.2143	0.8315	-0.1824	-1.2845
ULS: 3. D + (S or Lr or R)	0.0299	2.8223	0.0659	0.2549	-0.0557	-0.3807
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0804	5.5950	0.1772	0.6874	-0.1507	-1.0585
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0299	2.8223	0.0659	0.2549	-0.0557	-0.3807
ULS: 5b. D + 0.7E	0.0299	2.8223	0.0659	0.2549	-0.0557	-0.3807
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0804	5.5950	0.1772	0.6874	-0.1507	-1.0585
ULS: 8. 0.6D + 0.7E	0.0180	1.6934	0.0395	0.1529	-0.0334	-0.2284
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2735	7.8858	0.3563	1.2833	-1.8302	61.9564
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.2735	7.8858	0.3563	1.2833	-1.8302	61.9564
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4534	-1.2086	-0.1593	-0.5423	1.3214	-47.8459
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.9616	-0.6157	-0.1437	-0.4860	1.2385	-51.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1471	9.3926	0.3950	1.4587	-1.4816	45.6943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1471	9.3926	0.3950	1.4587	-1.4816	45.6943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6480	2.5718	0.0083	0.0895	0.8821	-36.6574
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2792	3.0165	0.0200	0.1318	0.8199	-39.0847
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1976	6.6199	0.2837	1.0262	-1.3866	46.3721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1976	6.6199	0.2837	1.0262	-1.3866	46.3721
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5976	-0.2009	-0.1030	-0.3430	0.9771	-35.9796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2287	0.2438	-0.0913	-0.3008	0.9149	-38.4069
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2855	6.7569	0.3299	1.1814	-1.8079	62.1087
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.2855	6.7569	0.3299	1.1814	-1.8079	62.1087
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4414	-2.3375	-0.1857	-0.6442	1.3437	-47.6936
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.9496	-1.7446	-0.1701	-0.5879	1.2608	-50.9299

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.6759
Shear X	-7.1724
Shear Z	0.6399
Moment X	2.3179
Moment Y (Twist)	3.0998
Moment Z	103.9751

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.3926
Shear X	-4.2855
Shear Z	0.3950
Moment X	1.4587
Moment Y (Twist)	1.8302
Moment Z	62.1087

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	-0.0286	3.3445	-0.0039	-0.0169	0.0256	0.4051
ULS: 2. D + L	-0.0286	3.3445	-0.0039	-0.0169	0.0256	0.4051
ULS: 3. D + (S or Lr or R)	-0.0929	8.2136	-0.0126	-0.0548	0.0831	1.2796
ULS: 3. D + (S or Lr or R)	-0.0286	3.3445	-0.0039	-0.0169	0.0256	0.4051
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0768	6.9963	-0.0104	-0.0454	0.0687	1.0610

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0286	3.3445	-0.0039	-0.0169	0.0256	0.4051
ULS: 5b. D + 0.7E	-0.0286	3.3445	-0.0039	-0.0169	0.0256	0.4051
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0768	6.9963	-0.0104	-0.0454	0.0687	1.0610
ULS: 8. 0.6D + 0.7E	-0.0172	2.0067	-0.0023	-0.0102	0.0154	0.2431
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.6214	10.0725	0.0104	0.0192	-0.1491	81.0152
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.6214	10.0725	0.0104	0.0192	-0.1491	81.0152
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4365	-2.0248	-0.0120	-0.0346	0.1412	-60.8036
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7330	-1.1909	-0.0211	-0.0677	0.2030	-64.2122
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2714	12.0423	0.0003	-0.0182	-0.0623	61.5185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2714	12.0423	0.0003	-0.0182	-0.0623	61.5185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2720	2.9694	-0.0164	-0.0586	0.1554	-44.8456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7443	3.5948	-0.0233	-0.0835	0.2018	-47.4020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2232	8.3905	0.0068	0.0102	-0.1054	60.8627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2232	8.3905	0.0068	0.0102	-0.1054	60.8627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3202	-0.6825	-0.0099	-0.0302	0.1123	-45.5014
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7926	-0.0570	-0.0168	-0.0550	0.1587	-48.0579
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.6100	8.7347	0.0119	0.0260	-0.1593	80.8531
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.6100	8.7347	0.0119	0.0260	-0.1593	80.8531
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4480	-3.3625	-0.0104	-0.0278	0.1309	-60.9657
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7444	-2.5286	-0.0196	-0.0610	0.1928	-64.3742

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	17.6585
Shear X	-9.3845
Shear Z	-0.0389
Moment X	-0.1278
Moment Y (Twist)	0.3636
Moment Z	136.5452

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	12.0423
Shear X	-5.6214
Shear Z	-0.0233
Moment X	-0.0835
Moment Y (Twist)	0.2030
Moment Z	81.0152

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0027	3.3076	0.0000	0.0000	0.0000	0.0885
ULS: 2. D + L	-0.0027	3.3076	0.0000	0.0000	0.0000	0.0885
ULS: 3. D + (S or Lr or R)	-0.0087	8.0939	0.0000	-0.0000	0.0001	0.2503
ULS: 3. D + (S or Lr or R)	-0.0027	3.3076	0.0000	0.0000	0.0000	0.0885
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0072	6.8973	0.0000	-0.0000	0.0001	0.2099
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0027	3.3076	0.0000	0.0000	0.0000	0.0885
ULS: 5b. D + 0.7E	-0.0027	3.3076	0.0000	0.0000	0.0000	0.0885
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0072	6.8973	0.0000	-0.0000	0.0001	0.2099
ULS: 8. 0.6D + 0.7E	-0.0016	1.9846	0.0000	0.0000	0.0000	0.0531
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.5903	9.9713	0.0000	-0.0000	0.0000	81.1468
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.5903	9.9713	0.0000	-0.0000	0.0000	81.1468
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4478	-1.9988	0.0000	-0.0000	0.0000	-61.3109
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7854	-1.2136	0.0000	0.0000	0.0000	-65.2936

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	-4.1979	11.8951	0.0000	-0.0000	0.0001	61.0036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.1979	11.8951	0.0000	-0.0000	0.0001	61.0036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3307	2.9175	0.0000	-0.0000	0.0001	-45.8397
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8339	3.5064	0.0000	-0.0000	0.0001	-48.8267
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.1934	8.3053	0.0000	-0.0000	0.0000	60.8822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.1934	8.3053	0.0000	-0.0000	0.0000	60.8822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3352	-0.6722	0.0000	0.0000	0.0000	-45.9610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.8384	-0.0833	0.0000	0.0000	0.0000	-48.9481
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.5893	8.6482	0.0000	-0.0000	0.0000	81.1114
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.5893	8.6482	0.0000	-0.0000	0.0000	81.1114
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4489	-3.3218	0.0000	-0.0000	0.0000	-61.3463
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7865	-2.5366	0.0000	0.0000	0.0000	-65.3290

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	17.4710
Shear X	-9.3207
Shear Z	0.0000
Moment X	-0.0004
Moment Y (Twist)	0.0010
Moment Z	136.6240

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	11.8951
Shear X	-5.5903
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	81.1468

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	-0.0286	3.3445	0.0039	0.0169	-0.0256	0.4051
ULS: 2. D + L	-0.0286	3.3445	0.0039	0.0169	-0.0256	0.4051
ULS: 3. D + (S or Lr or R)	-0.0929	8.2136	0.0126	0.0548	-0.0829	1.2796
ULS: 3. D + (S or Lr or R)	-0.0286	3.3445	0.0039	0.0169	-0.0256	0.4051
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0768	6.9963	0.0104	0.0453	-0.0685	1.0610
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0286	3.3445	0.0039	0.0169	-0.0256	0.4051
ULS: 5b. D + 0.7E	-0.0286	3.3445	0.0039	0.0169	-0.0256	0.4051
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0768	6.9963	0.0104	0.0453	-0.0685	1.0610
ULS: 8. 0.6D + 0.7E	-0.0172	2.0067	0.0023	0.0102	-0.0153	0.2431
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.6214	10.0725	-0.0104	-0.0192	0.1491	81.0152
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.6214	10.0725	-0.0104	-0.0192	0.1491	81.0152
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4365	-2.0248	0.0120	0.0346	-0.1411	-60.8036
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.7330	-1.1909	0.0211	0.0678	-0.2030	-64.2122
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2714	12.0423	-0.0003	0.0182	0.0625	61.5185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2714	12.0423	-0.0003	0.0182	0.0625	61.5185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2720	2.9694	0.0165	0.0585	-0.1552	-44.8456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7443	3.5948	0.0233	0.0834	-0.2016	-47.4020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2232	8.3905	-0.0068	-0.0102	0.1054	60.8627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2232	8.3905	-0.0068	-0.0102	0.1054	60.8627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3202	-0.6825	0.0099	0.0302	-0.1122	-45.5014
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7926	-0.0570	0.0168	0.0551	-0.1586	-48.0579

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.6100	8.7347	-0.0119	-0.0260	0.1593	80.8531
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.6100	8.7347	-0.0119	-0.0260	0.1593	80.8531
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4480	-3.3625	0.0104	0.0278	-0.1309	-60.9657
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.7444	-2.5286	0.0196	0.0610	-0.1928	-64.3742

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	17.6585
Shear X	-9.3846
Shear Z	0.0389
Moment X	0.1283
Moment Y (Twist)	0.3636
Moment Z	136.5453

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	12.0423
Shear X	-5.6214
Shear Z	0.0233
Moment X	0.0834
Moment Y (Twist)	0.2030
Moment Z	81.0152

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	0.0299	2.8222	-0.0659	-0.2549	0.0557	-0.3807
ULS: 2. D + L	0.0299	2.8222	-0.0659	-0.2549	0.0557	-0.3807
ULS: 3. D + (S or Lr or R)	0.0973	6.5192	-0.2144	-0.8319	0.1826	-1.2843
ULS: 3. D + (S or Lr or R)	0.0299	2.8222	-0.0659	-0.2549	0.0557	-0.3807
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0804	5.5950	-0.1772	-0.6877	0.1509	-1.0584
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0299	2.8222	-0.0659	-0.2549	0.0557	-0.3807
ULS: 5b. D + 0.7E	0.0299	2.8222	-0.0659	-0.2549	0.0557	-0.3807
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0804	5.5950	-0.1772	-0.6877	0.1509	-1.0584
ULS: 8. 0.6D + 0.7E	0.0180	1.6933	-0.0395	-0.1529	0.0334	-0.2284
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2735	7.8858	-0.3563	-1.2834	1.8302	61.9565
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.2735	7.8858	-0.3563	-1.2834	1.8302	61.9565
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4534	-1.2086	0.1593	0.5422	-1.3213	-47.8459
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.9616	-0.6157	0.1437	0.4859	-1.2385	-51.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1471	9.3926	-0.3950	-1.4590	1.4817	45.6945
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1471	9.3926	-0.3950	-1.4590	1.4817	45.6945
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6480	2.5718	-0.0083	-0.0898	-0.8819	-36.6572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2792	3.0165	-0.0200	-0.1320	-0.8198	-39.0845
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1976	6.6199	-0.2837	-1.0262	1.3866	46.3722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1976	6.6199	-0.2837	-1.0262	1.3866	46.3722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5975	-0.2009	0.1030	0.3430	-0.9771	-35.9796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.2287	0.2438	0.0913	0.3007	-0.9149	-38.4068
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2855	6.7569	-0.3299	-1.1814	1.8079	62.1087
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.2855	6.7569	-0.3299	-1.1814	1.8079	62.1087
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4414	-2.3375	0.1857	0.6442	-1.3436	-47.6936
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.9496	-1.7446	0.1701	0.5879	-1.2608	-50.9299

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	13.6759
Shear X	-7.1724
Shear Z	-0.6399
Moment X	-2.3183
Moment Y (Twist)	3.1008
Moment Z	103.9763

Result	Value (kip, kip-ft)
Axial	9.3926
Shear X	-4.2855
Shear Z	-0.3950
Moment X	-1.4590
Moment Y (Twist)	1.8302
Moment Z	62.1087

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: BT SIL530 40d 60inSquare - 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					
12	10in Pipe Sch 80	10.75	0.59					

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)

113	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.09,1.10,1.08,1.08,1.08,1.09,1.07,1.07,1.04,1.06,1.08,1.08,1.10,1.10,1.08,1.08,1.08,1.09	300	200	1
114	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.10,1.07,1.07,1.07,1.07,1.06,1.06,1.04,1.05,1.06,1.06,1.07,1.11,1.07,1.07,1.07,1.07	300	200	1
115	20	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.12,1.12,1.11,1.11,1.12,1.12,1.12,1.11,1.13,1.13,1.25,2.27,1.12,1.12,1.11,1.11,1.12,1.12,1.12,1.12	300	200	1
116	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.14,1.16,1.14,1.14,1.14,1.15,1.15,1.15,1.18,1.15,1.15,1.15,1.14,1.17,1.14,1.14,1.14,1.15	300	200	1
201	12	29.48	29.48	14.04	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.51,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
204	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.56,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.16,1.67,1.67,1.66,1.66	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,3.07,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.51,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,3.07,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
208	20	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,1.39,2.09,2.09,2.09,2.06,2.09,2.09,2.08,2.10,2.09,2.09,2.10,1.26,2.09,2.09,2.09,2.07	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.56,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.16,1.67,1.67,1.66,1.66	300	200	1
211	20	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.08,2.08,2.07,2.08,2.08,2.08,2.08,2.08,2.09,2.09,2.29,1.01,2.08,2.08,2.07,2.08,2.08,2.08,2.08	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.03,1.04,1.03,1.04,1.04,1.03,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.01,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
214	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.01,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.01,1.04,1.04,1.04,1.03	300	200	1
215	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.16,1.16,1.16,1.15,1.16,1.16,1.16,1.20,1.49,1.16,1.16,1.15,1.16,1.16,1.16,1.15,1.16	300	200	1
216	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.09,1.16,1.16,1.16,1.14,1.16,1.16,1.15,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15	300	200	1
301	12	29.48	29.48	14.04	-	300	200	1
302	6	1.30	1.30	2.00	-	300	200	1
303	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
304	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.59,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.36,1.67,1.67,1.66,1.66	300	200	1
305	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,2.04,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
306	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.23,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
307	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.71,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
308	20	1.33	1.33	2.05	1.98,1.99,1.98,1.99,1.99,1.98,1.85,1.85,1.76,1.64,1.84,1.84,1.80,1.92,1.91,1.91,2.13,2.02,1.86,1.86,1.76,1.56,1.83,1.83,1.81,1.94	300	200	1
309	3	2.60	2.60	4.00	-	300	200	1
310	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.53,1.67,1.67,1.66,1.65,1.67,1.67,1.69,1.67,1.67,1.67,1.66,1.15,1.67,1.67,1.66,1.66	300	200	1
311	20	1.33	1.33	2.05	1.85,1.85,1.85,1.85,1.85,1.85,1.58,1.58,1.46,1.47,1.57,1.57,1.52,1.51,1.67,1.67,2.25,1.15,1.59,1.59,1.46,1.47,1.56,1.56,1.53,1.52	300	200	1
312	6	1.30	1.30	2.00	-	300	200	1
313	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.09,1.10,1.08,1.08,1.08,1.09,1.07,1.07,1.04,1.06,1.08,1.08,1.10,1.10,1.08,1.08,1.08,1.09	300	200	1
314	20	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.10,1.07,1.07,1.07,1.07,1.06,1.06,1.04,1.05,1.06,1.06,1.07,1.11,1.07,1.07,1.07,1.07	300	200	1

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	31.65	6.46	56.26	44.91
114	159.30	97.43	31.88	6.46	56.26	44.91
115	159.30	75.13	21.34	6.46	56.26	44.91
116	159.30	75.13	21.94	6.46	56.26	44.91
201	851.50	419.76	229.67	229.67	255.45	255.45
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	30.81	6.46	56.26	44.91
214	159.30	97.43	30.83	6.46	56.26	44.91
215	159.30	75.13	22.18	6.46	56.26	44.91
216	159.30	75.13	20.65	6.46	56.26	44.91
301	851.50	419.76	229.67	229.67	255.45	255.45
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	31.65	6.46	56.26	44.91
314	159.30	97.43	31.87	6.46	56.26	44.91
315	159.30	75.13	21.63	6.46	56.26	44.91
316	159.30	75.13	21.12	6.46	56.26	44.91
401	851.50	419.76	229.67	229.67	255.45	255.45
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95

407	151.65	149.10	20.17	14.14	54.12	28.95
408	159.30	113.66	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	113.66	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	97.43	34.49	6.46	56.26	44.91
414	159.30	97.43	35.37	6.46	56.26	44.91
415	159.30	75.13	20.88	6.46	56.26	44.91
416	159.30	75.13	20.39	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	0.033	0.453	0.029	0.028	0.003	0.478	#13	0.492	Not Required	Pass
2	0.002	0.262	0.188	0.065	0.040	0.450	#13	0.054	Not Required	Pass
3	0.006	0.552	0.028	0.053	0.002	0.560	#13	0.046	Not Required	Pass
4	0.006	0.515	0.081	0.052	0.021	0.579	#13	0.082	Not Required	Pass
5	0.006	0.343	0.065	0.055	0.018	0.345	#13	0.076	Not Required	Pass
6	0.012	0.811	0.107	0.083	0.029	0.874	#13	0.046	Not Required	Pass
7	0.012	0.503	0.189	0.080	0.047	0.525	#13	0.076	Not Required	Pass
8	0.004	0.134	0.206	0.044	0.023	0.222	#24	0.102	Not Required	Pass
9	0.008	0.076	0.090	0.004	0.005	0.163	#13	0.206	Not Required	Pass
10	0.013	0.741	0.180	0.074	0.039	0.758	#13	0.082	Not Required	Pass
11	0.006	0.135	0.212	0.049	0.023	0.222	#23	0.102	Not Required	Pass
12	0.001	0.506	0.302	0.102	0.056	0.809	#13	0.054	Not Required	Pass
13	0.009	0.136	0.532	0.064	0.029	0.582	#21	0.306	Not Required	Pass
14	0.006	0.120	0.523	0.058	0.029	0.550	#24	0.204	Not Required	Pass
15	0.000	0.020	0.051	0.017	0.007	0.068	#21	Not Required	Not Required	Pass
16	0.000	0.019	0.051	0.016	0.007	0.067	#21	Not Required	Not Required	Pass
101	0.042	0.595	0.002	0.037	0.000	0.616	#13	0.492	Not Required	Pass
102	0.004	0.505	0.324	0.110	0.061	0.830	#13	0.036	Not Required	Pass
103	0.011	0.876	0.070	0.087	0.010	0.916	#13	0.046	Not Required	Pass
104	0.011	0.847	0.193	0.084	0.042	0.919	#13	0.082	Not Required	Pass
105	0.011	0.544	0.201	0.087	0.052	0.570	#13	0.076	Not Required	Pass
106	0.011	0.894	0.069	0.089	0.010	0.927	#13	0.046	Not Required	Pass
107	0.011	0.556	0.193	0.089	0.050	0.582	#13	0.076	Not Required	Pass
108	0.005	0.059	0.197	0.048	0.023	0.233	#21	0.102	Not Required	Pass
109	0.018	0.068	0.061	0.001	0.000	0.133	#13	0.206	Not Required	Pass
110	0.011	0.847	0.185	0.084	0.040	0.909	#13	0.082	Not Required	Pass
111	0.006	0.077	0.205	0.050	0.023	0.233	#21	0.102	Not Required	Pass
112	0.004	0.513	0.333	0.110	0.064	0.846	#13	0.036	Not Required	Pass
113	0.009	0.235	0.547	0.069	0.030	0.715	#21	0.306	Not Required	Pass
114	0.009	0.260	0.541	0.067	0.030	0.725	#21	0.306	Not Required	Pass
115	0.011	0.450	0.286	0.054	0.023	0.653	#21	0.507	Not Required	Pass
116	0.005	0.407	0.284	0.053	0.023	0.622	#21	0.507	Not Required	Pass
201	0.042	0.595	0.000	0.036	0.000	0.616	#13	0.492	Not Required	Pass
202	0.003	0.505	0.327	0.109	0.062	0.832	#13	0.036	Not Required	Pass
203	0.011	0.889	0.069	0.088	0.010	0.926	#13	0.046	Not Required	Pass
204	0.011	0.826	0.183	0.082	0.040	0.889	#13	0.082	Not Required	Pass
205	0.011	0.552	0.101	0.088	0.040	0.576	#13	0.076	Not Required	Pass

205	0.011	0.552	0.191	0.088	0.049	0.576	#13	0.076	Not Required	Pass
206	0.011	0.889	0.069	0.088	0.010	0.926	#13	0.046	Not Required	Pass
207	0.011	0.552	0.191	0.088	0.049	0.576	#13	0.076	Not Required	Pass
208	0.005	0.057	0.196	0.049	0.023	0.232	#21	0.102	Not Required	Pass
209	0.017	0.065	0.061	0.001	0.000	0.130	#13	0.206	Not Required	Pass
210	0.011	0.826	0.183	0.082	0.040	0.889	#13	0.082	Not Required	Pass
211	0.006	0.066	0.202	0.053	0.023	0.241	#21	0.102	Not Required	Pass
212	0.003	0.505	0.326	0.109	0.062	0.832	#13	0.036	Not Required	Pass
213	0.009	0.277	0.518	0.067	0.029	0.728	#21	0.306	Not Required	Pass
214	0.009	0.270	0.510	0.063	0.029	0.709	#21	0.306	Not Required	Pass
215	0.011	0.327	0.287	0.053	0.023	0.551	#21	0.507	Not Required	Pass
216	0.006	0.287	0.283	0.049	0.023	0.523	#21	0.507	Not Required	Pass
301	0.042	0.595	0.002	0.037	0.000	0.616	#13	0.492	Not Required	Pass
302	0.004	0.513	0.333	0.110	0.064	0.846	#13	0.036	Not Required	Pass
303	0.011	0.894	0.069	0.089	0.010	0.927	#13	0.046	Not Required	Pass
304	0.011	0.847	0.185	0.084	0.040	0.909	#13	0.082	Not Required	Pass
305	0.011	0.556	0.192	0.089	0.050	0.582	#13	0.076	Not Required	Pass
306	0.011	0.876	0.070	0.087	0.010	0.916	#13	0.046	Not Required	Pass
307	0.011	0.544	0.201	0.087	0.052	0.570	#13	0.076	Not Required	Pass
308	0.004	0.089	0.219	0.053	0.023	0.241	#21	0.102	Not Required	Pass
309	0.018	0.068	0.061	0.001	0.000	0.133	#13	0.206	Not Required	Pass
310	0.011	0.847	0.193	0.084	0.042	0.919	#13	0.082	Not Required	Pass
311	0.006	0.111	0.224	0.054	0.023	0.235	#21	0.102	Not Required	Pass
312	0.004	0.505	0.324	0.110	0.061	0.830	#13	0.036	Not Required	Pass
313	0.009	0.236	0.547	0.069	0.030	0.715	#21	0.306	Not Required	Pass
314	0.009	0.260	0.541	0.067	0.030	0.725	#21	0.306	Not Required	Pass
315	0.011	0.331	0.287	0.050	0.023	0.554	#21	0.507	Not Required	Pass
316	0.006	0.288	0.283	0.048	0.023	0.523	#21	0.507	Not Required	Pass
401	0.033	0.453	0.029	0.028	0.003	0.478	#13	0.492	Not Required	Pass
402	0.001	0.506	0.302	0.102	0.056	0.809	#13	0.054	Not Required	Pass
403	0.012	0.811	0.107	0.083	0.029	0.874	#13	0.046	Not Required	Pass
404	0.013	0.741	0.180	0.074	0.039	0.758	#13	0.082	Not Required	Pass
405	0.012	0.503	0.189	0.080	0.047	0.525	#13	0.076	Not Required	Pass
406	0.006	0.552	0.028	0.053	0.002	0.560	#13	0.046	Not Required	Pass
407	0.006	0.343	0.065	0.055	0.018	0.345	#13	0.076	Not Required	Pass
408	0.000	0.019	0.051	0.016	0.007	0.067	#21	Not Required	Not Required	Pass
409	0.008	0.076	0.090	0.004	0.005	0.163	#13	0.206	Not Required	Pass
410	0.006	0.515	0.081	0.052	0.021	0.579	#13	0.082	Not Required	Pass
411	0.000	0.020	0.051	0.017	0.007	0.068	#21	Not Required	Not Required	Pass
412	0.002	0.262	0.188	0.065	0.040	0.450	#13	0.054	Not Required	Pass
413	0.009	0.136	0.531	0.064	0.029	0.582	#21	0.204	Not Required	Pass
414	0.006	0.120	0.523	0.058	0.029	0.550	#24	0.306	Not Required	Pass
415	0.011	0.461	0.287	0.049	0.023	0.663	#21	0.507	Not Required	Pass
416	0.005	0.429	0.282	0.044	0.023	0.634	#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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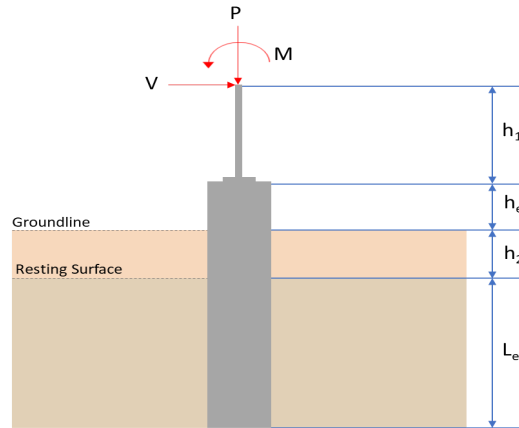
SkyCiv Foundation Design

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

$L = 8.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.393	13.676
V_x (kip)	-4.285	-7.172
V_z (kip)	0.395	0.640
M_x (kipft)	1.459	2.318
M_z (kipft)	62.109	103.975

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,

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

$$H_o = \frac{(-4.285 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68232 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(62.109 \text{ kipft}) + ((-4.285 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.89 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.788 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.395 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.062898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.459 \text{ kipft}) + ((0.395 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23232 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.1196 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.788 \text{ ft}), (3.1196 \text{ ft})]$$

$$L_{e,req} = 7.788 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.788 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.91624$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.393 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58706 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58706 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29353$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.68232 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.89 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.89 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.89 \text{ kipft/ft})) + (4 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8658 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.89 \text{ kipft/ft})) + (3 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.89 \text{ kipft/ft})) + (2 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.28211 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.89 \text{ kipft/ft})) + ((-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.161 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8658 \text{ ft})}{2}$$

$$p_a = 0.43993 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28211 \text{ kip/ft}^2)}{(0.43993 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64126$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.161 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91058$$

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

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

Considering z-direction:

$H_o = 0.062898 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.23232 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23232 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.062898 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.23232 \text{ kipft/ft})) + (4 \times (0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0955 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.23232 \text{ kipft/ft})) + (3 \times (0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.23232 \text{ kipft/ft})) + (2 \times (0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.037715 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.23232 \text{ kipft/ft})) + ((0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.082985 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0955 \text{ ft})}{2}$$

$$p_a = 0.45716 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.037715 \text{ kip/ft}^2)}{(0.45716 \text{ kip/ft}^2)}$$

$$Ratio = 0.082498$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

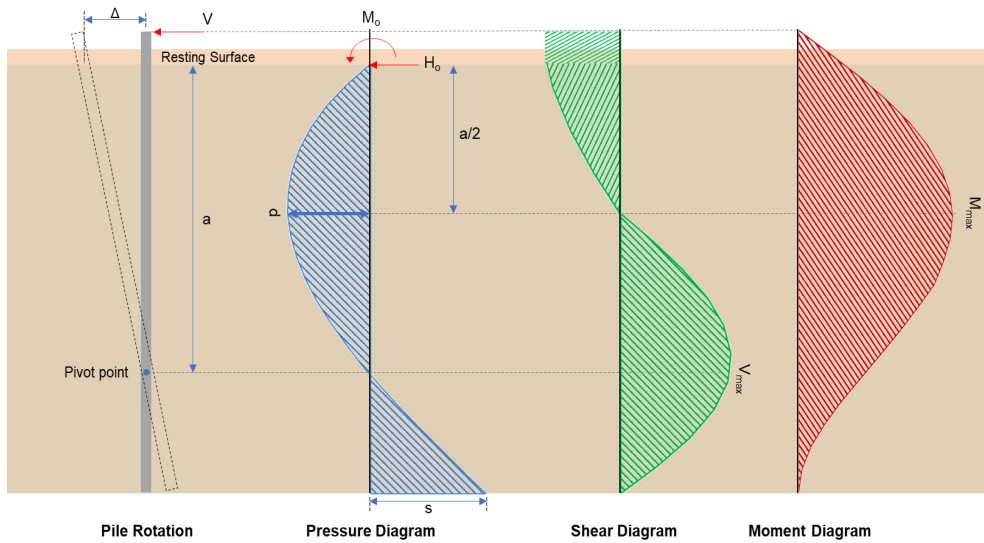
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.082985 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.065087$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.172 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.142 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.97 \text{ kipft}) + ((-7.172 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.557 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.557 \text{ kipft/ft})}{(-1.142 \text{ kip/ft})}$$

$$E = 14.497 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.557 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times 16.557) + (4 \times (-1.142) \times 8.5)}$$

$$a = \frac{(6 \times (16.557 \text{ kipft/ft})) + (4 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft}))}{(6 \times (16.557 \text{ kipft/ft})) + (4 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8657 \text{ ft}$$

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

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

$$V_{max} = ((-1.142 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8657 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8657 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.8 \text{ kip}$$

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

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

$$M_{max} = ((-1.142 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(14.497 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.873 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.64 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10191 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.318 \text{ kipft}) + ((0.64 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.36911 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.36911 \text{ kipft/ft})}{(0.10191 \text{ kip/ft})}$$

$$E = 3.6219 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.36911 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.10191 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.36911 \text{ kipft/ft})) + (4 \times (0.10191 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0988 \text{ ft}$$

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

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

$$V_{max} = ((0.10191 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0988 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0988 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.57963 \text{ kip}$$

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

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

$$M_{max} = ((0.10191 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(3.6219 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.155 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{13.676 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.142 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.142 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(13.676 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0051122$</p>	<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> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.676 \text{ kip} \rightarrow 13676 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13676 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.31 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.31 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.31 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.31 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

$V_{max} = 16.8 \text{ kip}$ - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(16.8 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.15096$$

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

Considering z-direction:

$V_{max} = 0.57963 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.57963 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.0052087$$

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:

$M_{max} = 67.873 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.873 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27193$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 2.155 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(2.155 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0086339$$

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

REFERENCES	CALCULATIONS	RESULTS
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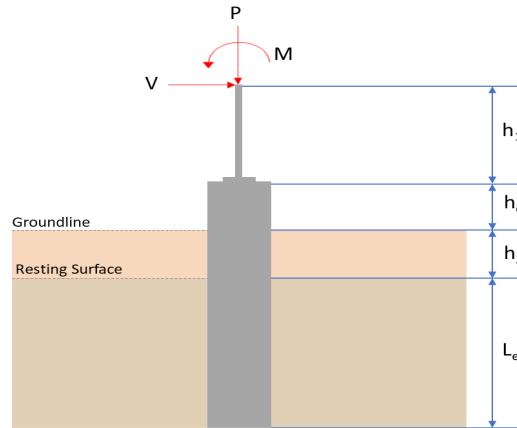
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.393	13.676
V_x (kip)	-4.285	-7.172
V_z (kip)	-0.395	-0.640
M_x (kipft)	-1.459	-2.318
M_z (kipft)	62.109	103.976

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,

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

$$H_o = \frac{(-4.285 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68232 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(62.109 \text{ kipft}) + ((-4.285 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.89 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.788 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.395 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.062898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.459 \text{ kipft}) + ((-0.395 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23232 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1799 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.788 \text{ ft}), (2.1799 \text{ ft})]$$

$$L_{e,req} = 7.788 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.788 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.91624$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.393 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58706 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58706 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29353$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.68232 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.89 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.89 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.89 \text{ kipft/ft})) + (4 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8658 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.89 \text{ kipft/ft})) + (3 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.89 \text{ kipft/ft})) + (2 \times (-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.28211 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.89 \text{ kipft/ft})) + ((-0.68232 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.161 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8658 \text{ ft})}{2}$$

$$p_a = 0.43993 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28211 \text{ kip/ft}^2)}{(0.43993 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64126$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.161 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91058$$

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

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

Considering z-direction:

$H_o = -0.062898 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.23232 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23232 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.062898 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.23232 \text{ kipft/ft})) + (4 \times (-0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0955 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.23232 \text{ kipft/ft})) + (3 \times (-0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.23232 \text{ kipft/ft})) + (2 \times (-0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.012689 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.23232 \text{ kipft/ft})) + ((-0.062898 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.0058118 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0955 \text{ ft})}{2}$$

$$p_a = 0.45716 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.012689 \text{ kip/ft}^2)}{(0.45716 \text{ kip/ft}^2)}$$

$$Ratio = -0.027756$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

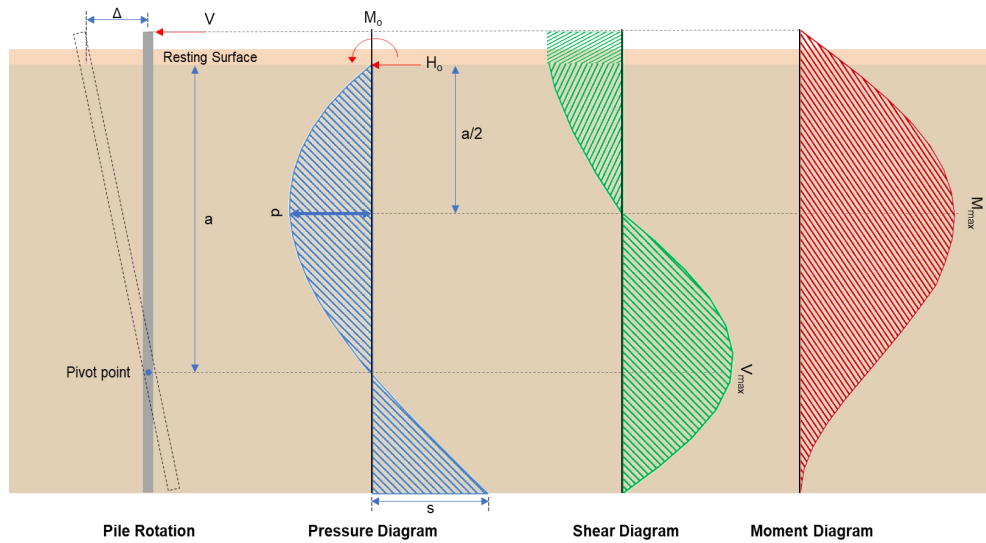
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0058118 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = -0.0045583$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.172 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.142 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(103.98 \text{ kipft}) + ((-7.172 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.557 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.557 \text{ kipft/ft})}{(-1.142 \text{ kip/ft})}$$

$$E = 14.497 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.557 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (16.557 \text{ kipft/ft})) + (4 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = \frac{(6 \times (16.557 \text{ kipft/ft})) + (4 \times (-1.142 \text{ kip/ft}) \times (8.5 \text{ ft}))}{}$$

$$a = 5.8657 \text{ ft}$$

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

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

$$V_{max} = ((-1.142 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8657 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8657 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.8 \text{ kip}$$

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

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

$$M_{max} = ((-1.142 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(14.497 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.497 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8657 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 67.874 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.64 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10191 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.318 \text{ kipft}) + ((-0.64 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.36911 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.36911 \text{ kipft/ft})}{(-0.10191 \text{ kip/ft})}$$

$$E = 3.6219 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.36911 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.10191 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.36911 \text{ kipft/ft})) + (4 \times (-0.10191 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0988 \text{ ft}$$

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

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

$$V_{max} = ((-0.10191 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0988 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0988 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.57963 \text{ kip}$$

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

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

$$M_{max} = ((-0.10191 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(3.6219 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6219 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0988 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.155 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.676 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.142 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.142 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.676 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0051122$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.676 \text{ kip} \rightarrow 13676 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13676 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.31 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.31 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.31 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.31 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.28 \text{ kip}$$

Considering x-direction:

$V_{max} = 16.8 \text{ kip}$ - Maximum shear force in the x-direction,
 Ratio - Capacity

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

$$Ratio = \frac{(16.8 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.15097$$

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

Considering z-direction:

$V_{max} = 0.57963 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.57963 \text{ kip})}{(111.28 \text{ kip})}$$

$$Ratio = 0.0052087$$

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:

$M_{max} = 67.874 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(67.874 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.27193$$

Status: **PASS**
Ratio: **0.270**

Considering z-direction:

$M_{max} = 2.155 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(2.155 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0086339$$

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

REFERENCES	CALCULATIONS	RESULTS
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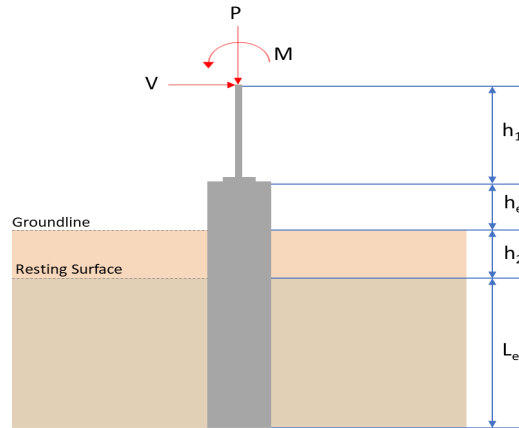
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	12.042	17.658
V_x (kip)	-5.621	-9.385
V_z (kip)	-0.023	-0.039
M_x (kipft)	-0.083	-0.128
M_z (kipft)	81.015	136.545

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.621 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.89506 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(81.015 \text{ kipft}) + ((-5.621 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.9 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 8.3554 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0036624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.083 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013217 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.94712 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(8.3554 \text{ ft}), (0.94712 \text{ ft})]$$

$$L_{e,req} = 8.355 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(8.355 \text{ ft})}{(9 \text{ ft})}$$

$$Ratio = 0.92833$$

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

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

$$q = \frac{(12.042 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.75262 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.75262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37631$$

Status: **PASS**
Ratio: **0.380**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.89506 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.9 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.9 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.9 \text{ kipft/ft})) + (4 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2204 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.9 \text{ kipft/ft})) + (3 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.9 \text{ kipft/ft})) + (2 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.30851 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.9 \text{ kipft/ft})) + ((-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.3145 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2204 \text{ ft})}{2}$$

$$p_a = 0.46653 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30851 \text{ kip/ft}^2)}{(0.46653 \text{ kip/ft}^2)}$$

$$Ratio = 0.66128$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.3145 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.97368$$

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

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

Considering z-direction:

$H_o = -0.0036624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.013217 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013217 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0036624 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.013217 \text{ kipft/ft})) + (4 \times (-0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4683 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.013217 \text{ kipft/ft})) + (3 \times (-0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 [(3 \times (0.013217 \text{ kipft/ft})) + (2 \times (-0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = -0.00074633 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.013217 \text{ kipft/ft})) + ((-0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = -0.0004836 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4683 \text{ ft})}{2}$$

$$p_a = 0.48512 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00074633 \text{ kip/ft}^2)}{(0.48512 \text{ kip/ft}^2)}$$

$$Ratio = -0.0015384$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

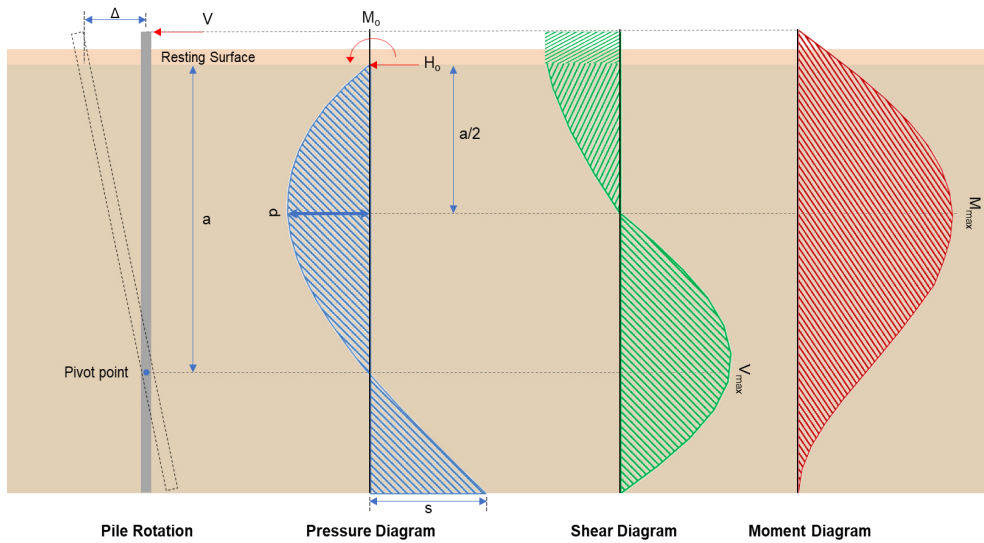
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0004836 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = -0.00035823$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.385 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.4944 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(136.54 \text{ kipft}) + ((-9.385 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 21.743 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(21.743 \text{ kipft/ft})}{(-1.4944 \text{ kip/ft})}$$

$$E = 14.549 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (21.743 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (21.743 \text{ kipft/ft})) + (4 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = \frac{(6 \times (21.743 \text{ kipft/ft})) + (4 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.219 \text{ ft}$$

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

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

$$V_{max} = ((-1.4944 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.219 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.219 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.041 \text{ kip}$$

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

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

$$M_{max} = ((-1.4944 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(14.549 \text{ ft})}{(9 \text{ ft})} + \frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 89.806 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.039 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0062102 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.128 \text{ kipft}) + ((-0.039 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.020382 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.020382 \text{ kipft/ft})}{(-0.0062102 \text{ kip/ft})}$$

$$E = 3.2821 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.020382 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0062102 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.020382 \text{ kipft/ft})) + (4 \times (-0.0062102 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4848 \text{ ft}$$

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

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

$$V_{max} = ((-0.0062102 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4848 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4848 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.032661 \text{ kip}$$

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

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

$$M_{max} = ((-0.0062102 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.2821 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12711 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{17.658 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.009 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.009 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(17.658 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0066007$</p>	<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> <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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.658 \text{ kip} \rightarrow 17658 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(17658 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

22.5.5.1.2 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} = 21.041 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.041 \text{ kip})}{(111.63 \text{ kip})}$$

$$Ratio = 0.1885$$

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

Considering z-direction:

$V_{max} = 0.032661 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.032661 \text{ kip})}{(111.63 \text{ kip})}$$

$$Ratio = 0.00029259$$

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:

$M_{max} = 89.806 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(89.806 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.3598$$

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

Considering z-direction:

$M_{max} = 0.12711 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.12711 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00050927$$

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

REFERENCES	CALCULATIONS	RESULTS
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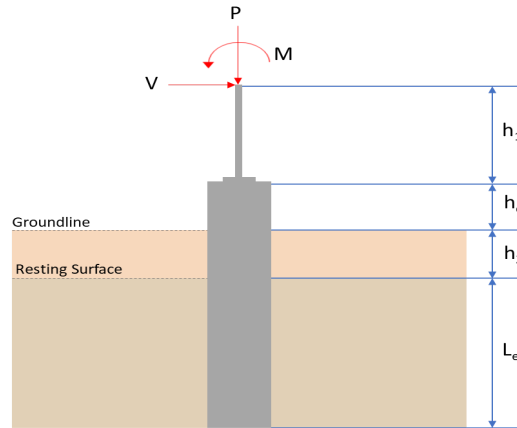
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.895	17.471
V_x (kip)	-5.590	-9.321
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	81.147	136.624

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.59 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.89013 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(81.147 \text{ kipft}) + ((-5.59 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.921 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 8.3712 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.3712 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 8.371 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.371 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.93011$$

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

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

$$q = \frac{(11.895 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.74344 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.74344 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37172$$

Status: **PASS**
Ratio: **0.370**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.89013 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.921 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.921 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.89013 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.921 \text{ kipft/ft})) + (4 \times (-0.89013 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2193 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.921 \text{ kipft/ft})) + (3 \times (-0.89013 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.921 \text{ kipft/ft})) + (2 \times (-0.89013 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.31133 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.921 \text{ kipft/ft})) + ((-0.89013 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.3209 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2193 \text{ ft})}{2}$$

$$p_a = 0.46645 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.31133 \text{ kip/ft}^2)}{(0.46645 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66743$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

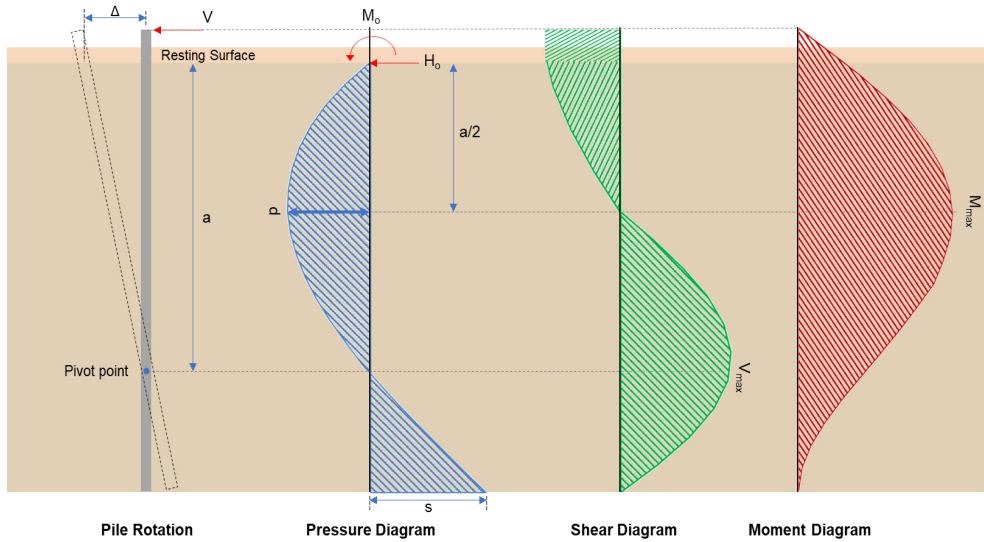
$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.3209 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.321 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.4842 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(136.62 \text{ kipft}) + ((-9.321 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 21.755 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(21.755 \text{ kipft/ft})}{(-1.4842 \text{ kip/ft})}$$

$$E = 14.658 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (21.755 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.4842 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (21.755 \text{ kipft/ft})) + (4 \times (-1.4842 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2178 \text{ ft}$$

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

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

$$V_{max} = ((-1.4842 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.658 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2178 \text{ ft})}{(9 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (14.658 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2178 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.024 \text{ kip}$$

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

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

$$M_{max} = ((-1.4842 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(14.658 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2178 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.658 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2178 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (14.658 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2178 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 89.763 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{17.471 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.016 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.016 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$Ratio = \frac{(17.471 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0065308$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 b_w = 48 in - Effective width,
 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.471 \text{ kip} \rightarrow 17471 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(17471 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.81 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.81 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.81 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((120.81 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.61 \text{ kip}$$

Considering x-direction:

$V_{max} = 21.024 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(21.024 \text{ kip})}{(111.61 \text{ kip})}$$

$$\text{Ratio} = 0.18837$$

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

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: $M_{max} = 89.763 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(89.763 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.35963$$

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

REFERENCES	CALCULATIONS	RESULTS
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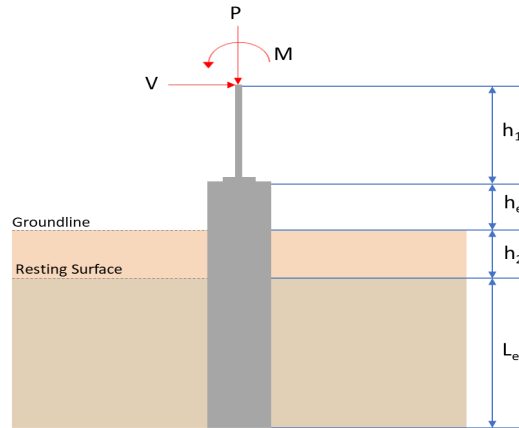
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 9$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	12.042	17.658
V_x (kip)	-5.621	-9.385
V_z (kip)	0.023	0.039
M_x (kipft)	0.083	0.128
M_z (kipft)	81.015	136.545

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.621 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.89506 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(81.015 \text{ kipft}) + ((-5.621 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.9 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 8.3554 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.023 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0036624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.083 \text{ kipft}) + ((0.023 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013217 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0906 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.3554 \text{ ft}), (1.0906 \text{ ft})]$$

$$L_{e,req} = 8.355 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.355 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.92833$$

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

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

$$q = \frac{(12.042 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.75262 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.75262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37631$$

Status: **PASS**
Ratio: **0.380**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.89506 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 12.9 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.9 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (12.9 \text{ kipft/ft})) + (4 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2204 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (12.9 \text{ kipft/ft})) + (3 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (12.9 \text{ kipft/ft})) + (2 \times (-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.30851 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (12.9 \text{ kipft/ft})) + ((-0.89506 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.3145 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2204 \text{ ft})}{2}$$

$$p_a = 0.46653 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30851 \text{ kip/ft}^2)}{(0.46653 \text{ kip/ft}^2)}$$

$$Ratio = 0.66128$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.3145 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.97368$$

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

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

Considering z-direction:

$H_o = 0.0036624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.013217 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013217 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0036624 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.013217 \text{ kipft/ft})) + (4 \times (0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4683 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.013217 \text{ kipft/ft})) + (3 \times (0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.013217 \text{ kipft/ft})) + (2 \times (0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.0020197 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.013217 \text{ kipft/ft})) + ((0.0036624 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 0.0043996 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4683 \text{ ft})}{2}$$

$$p_a = 0.48512 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0020197 \text{ kip/ft}^2)}{(0.48512 \text{ kip/ft}^2)}$$

$$Ratio = 0.0041633$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

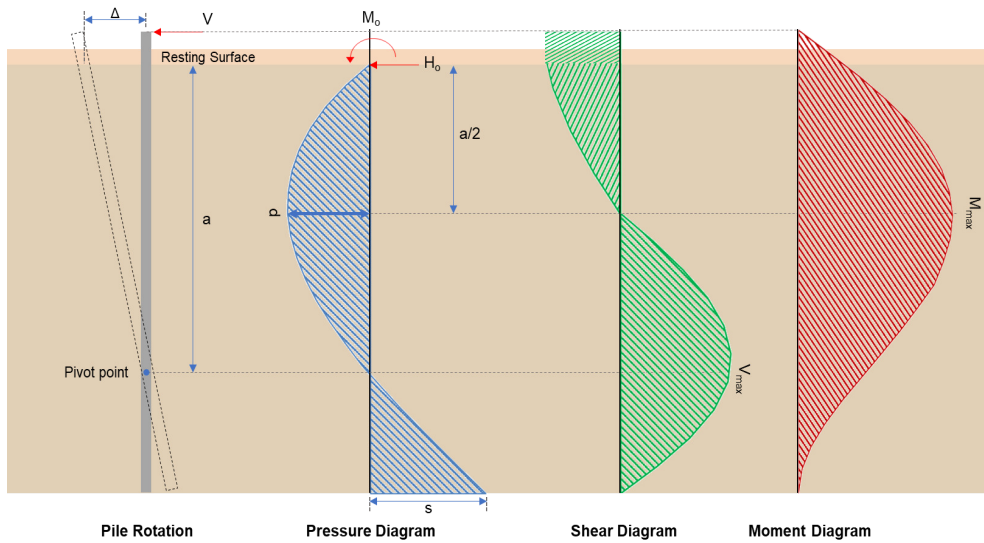
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0043996 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.003259$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.385 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.4944 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(136.54 \text{ kipft}) + ((-9.385 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 21.743 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(21.743 \text{ kipft/ft})}{(-1.4944 \text{ kip/ft})}$$

$$E = 14.549 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (21.743 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (21.743 \text{ kipft/ft})) + (4 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = \frac{(6 \times (21.743 \text{ kipft/ft})) + (4 \times (-1.4944 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.219 \text{ ft}$$

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

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

$$V_{max} = ((-1.4944 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.219 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.219 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.041 \text{ kip}$$

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

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

$$M_{max} = ((-1.4944 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(14.549 \text{ ft})}{(9 \text{ ft})} + \frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.549 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.219 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 89.806 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.039 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0062102 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.128 \text{ kipft}) + ((0.039 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.020382 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.020382 \text{ kipft/ft})}{(0.0062102 \text{ kip/ft})}$$

$$E = 3.2821 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.020382 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0062102 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.020382 \text{ kipft/ft})) + (4 \times (0.0062102 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4848 \text{ ft}$$

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

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

$$V_{max} = ((0.0062102 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4848 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4848 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.032661 \text{ kip}$$

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

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

$$M_{max} = ((0.0062102 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(3.2821 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2821 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.4848 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12711 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.658 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.009 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.009 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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;">$Ratio = \frac{(17.658 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0066007$</p>	<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> <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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.658 \text{ kip} \rightarrow 17658 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(17658 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

22.5.5.1.2 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} = 21.041 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.041 \text{ kip})}{(111.63 \text{ kip})}$$

$$Ratio = 0.1885$$

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

Considering z-direction:

$V_{max} = 0.032661 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.032661 \text{ kip})}{(111.63 \text{ kip})}$$

$$Ratio = 0.00029259$$

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:

$M_{max} = 89.806 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(89.806 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.3598$$

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

Considering z-direction:

$M_{max} = 0.12711 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.12711 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00050927$$

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