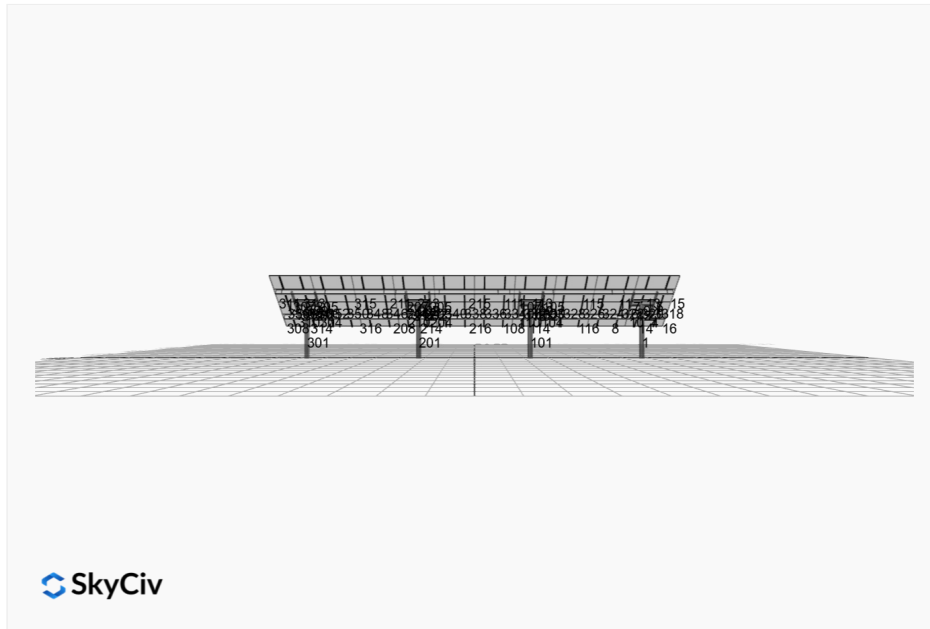


Project Name: MTSOLAR_17FCL6K2C4L0K **Date:** Thu Aug 21 2025
Location: Stratham, NH, USA **Number of Modules:** 40
Unique ID: 4P-19.75-8TOP-XD-12-L-4Hx10W-F89E **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	69.67 ft
Winter Tilt Angle	35
Front Edge Clearance	5 ft

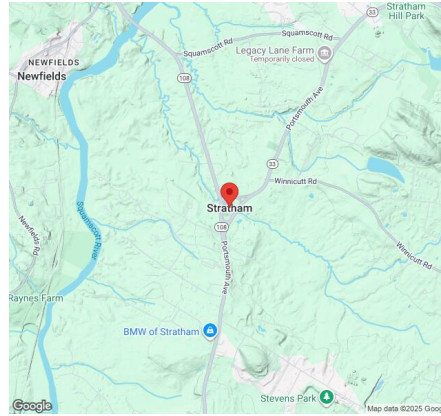
MT Solar Bill of Materials (4P-19.75-8TOP-XD-12-L-4Hx10W-F89E)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-HOOK-4PK	Hook Clamp	10

Rail Bill of Materials

Part	Qty
Rails (180in)	20
Rail Attachment	40
Module Mid Clamp	60
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: Stratham, NH, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	4
Number of Columns:	10
Total Number of Modules:	40
Winter Tilt Angle:	35
Front Edge Clearance:	5
Total Array Height at Tilt:	13.62 ft
Total Frame Length:	68.75 ft
Module Info/Notes:	Silfab SIL-530XM
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	69.67 ft
Rail Length:	180.40 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	9.31 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.50 ft Pile 3: 8.50 ft Pile 4: 8.00 ft
Foundation Volume:	8.639 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Stratham, NH, USA
Wind Speed:	107 mph

Snow Load:

50 psf

Design Disclaimer

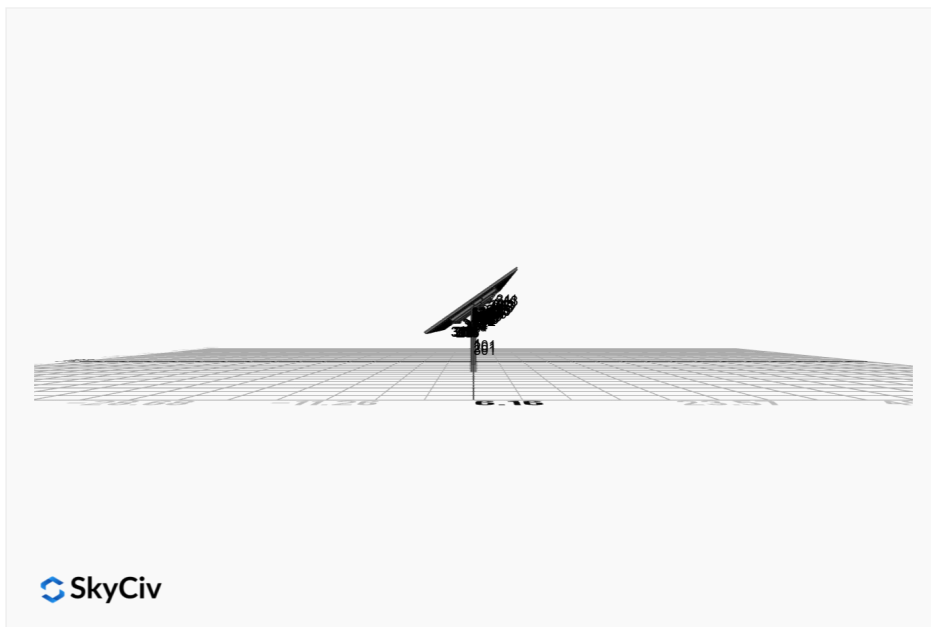
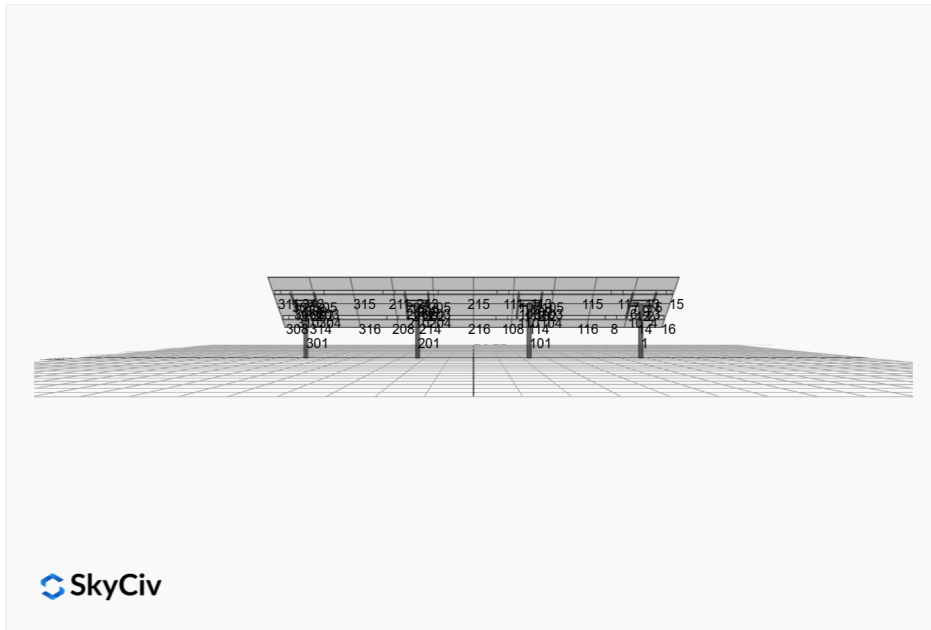
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

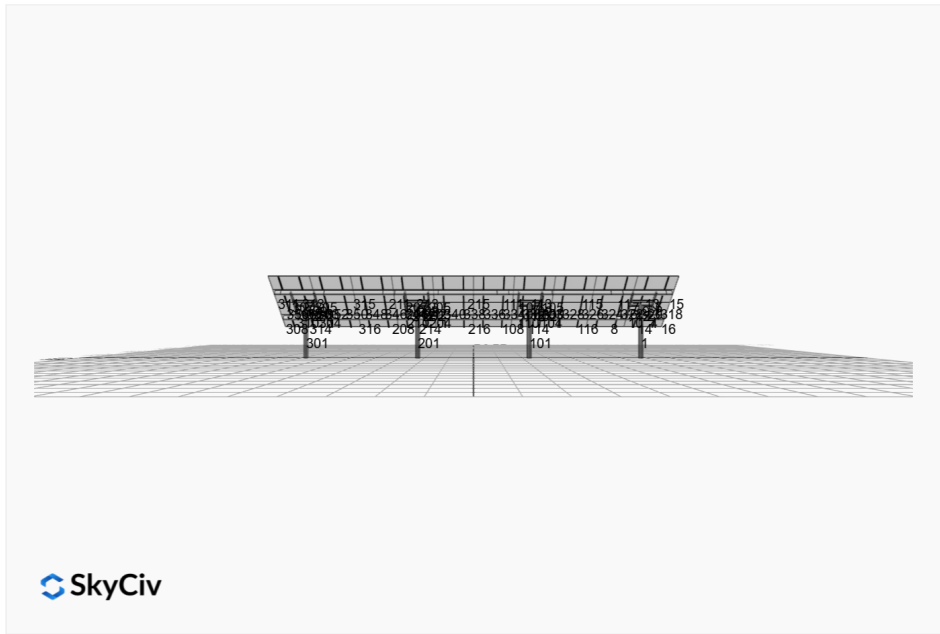
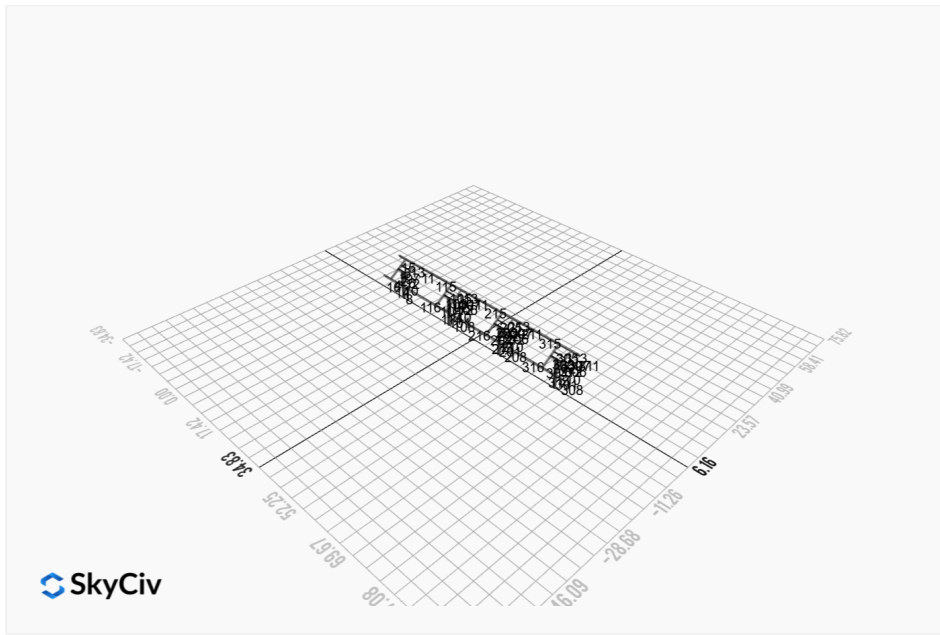
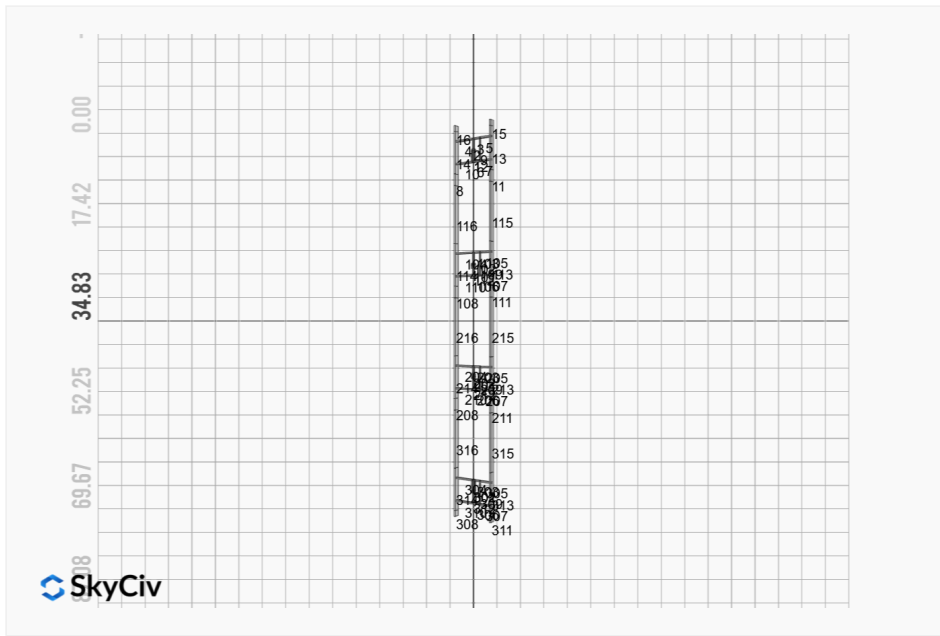
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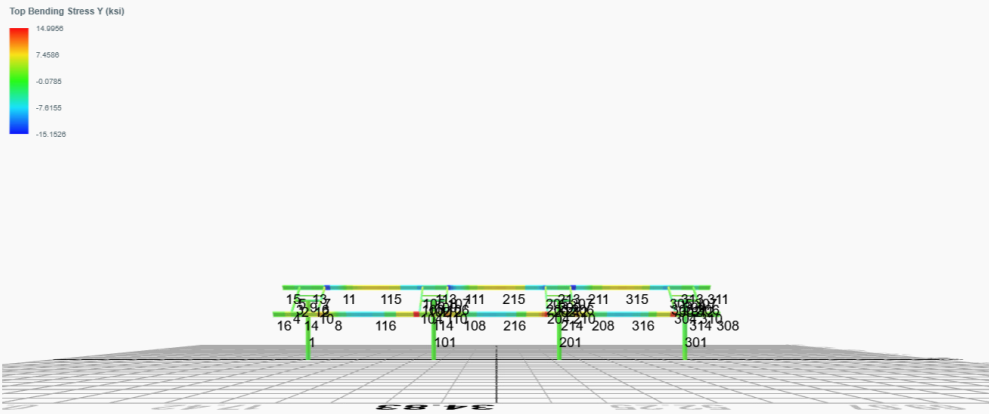
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Design Notes:

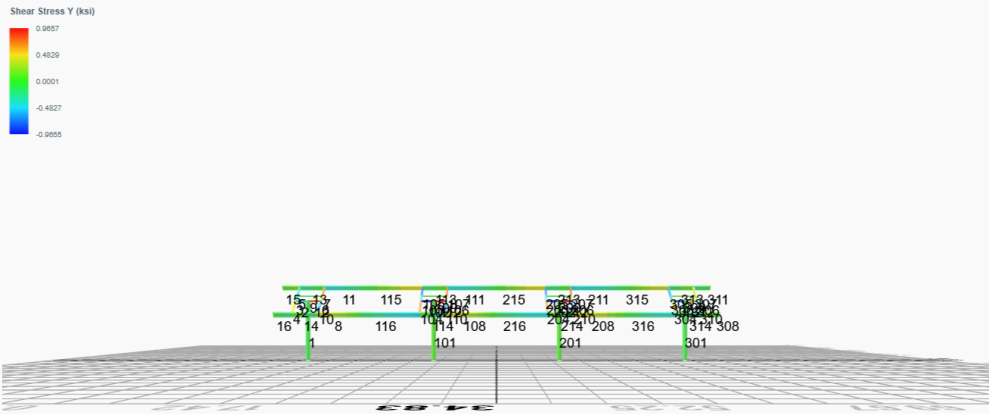
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



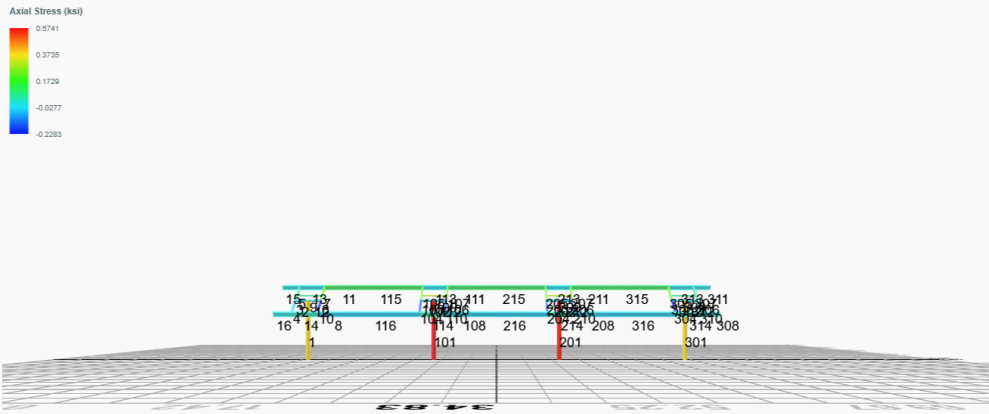




SkyCiv



SkyCiv



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0363	1.8520	0.0950	0.2404	-0.0616	-0.2896
ULS: 2. D + L	0.0363	1.8520	0.0950	0.2404	-0.0616	-0.2896
ULS: 3. D + (S or Lr or R)	0.1281	5.1763	0.3366	0.8532	-0.2190	-1.0833
ULS: 3. D + (S or Lr or R)	0.0363	1.8520	0.0950	0.2404	-0.0616	-0.2896
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1051	4.3453	0.2762	0.7000	-0.1796	-0.8849
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0363	1.8520	0.0950	0.2404	-0.0616	-0.2896
ULS: 5b. D + 0.7E	0.0363	1.8520	0.0950	0.2404	-0.0616	-0.2896
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1051	4.3453	0.2762	0.7000	-0.1796	-0.8849
ULS: 8. 0.6D + 0.7E	0.0218	1.1112	0.0570	0.1443	-0.0369	-0.1737
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8779	4.5386	0.3657	0.8803	-0.7723	18.1466
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8779	4.5386	0.3657	0.8803	-0.7723	18.1466
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6492	-0.4129	-0.1295	-0.2901	0.5285	-15.3924
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4034	-0.0496	-0.1174	-0.2608	0.5056	-18.6504
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3305	6.3602	0.4792	1.1800	-0.7127	12.9422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3305	6.3602	0.4792	1.1800	-0.7127	12.9422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3148	2.6465	0.1078	0.3021	0.2629	-12.2120
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1305	2.9190	0.1169	0.3241	0.2457	-14.6555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3994	3.8670	0.2980	0.7203	-0.5947	13.5375
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3994	3.8670	0.2980	0.7203	-0.5947	13.5375
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2459	0.1533	-0.0734	-0.1575	0.3810	-11.6167
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0616	0.4258	-0.0643	-0.1355	0.3638	-14.0602
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8924	3.7978	0.3277	0.7842	-0.7477	18.2624
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8924	3.7978	0.3277	0.7842	-0.7477	18.2624
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6346	-1.1537	-0.1675	-0.3863	0.5531	-15.2766
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3889	-0.7905	-0.1555	-0.3570	0.5302	-18.5346

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	9.7808
Shear X	-3.1903
Shear Z	0.7306
Moment X	1.8185
Moment Y (Twist)	1.3434
Moment Z	31.6468

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	6.3602
Shear X	-1.8924
Shear Z	0.4792
Moment X	1.1800
Moment Y (Twist)	0.7723
Moment Z	18.6504

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	-0.0363	2.4444	-0.0080	-0.0210	0.0183	0.3461
ULS: 2. D + L	-0.0363	2.4444	-0.0080	-0.0210	0.0183	0.3461
ULS: 3. D + (S or Lr or R)	-0.1281	7.2663	-0.0283	-0.0739	0.0645	1.1764
ULS: 3. D + (S or Lr or R)	-0.0363	2.4444	-0.0080	-0.0210	0.0183	0.3461
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1051	6.0608	-0.0232	-0.0607	0.0529	0.9689

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0363	2.4444	-0.0080	-0.0210	0.0183	0.3461
ULS: 5b. D + 0.7E	-0.0363	2.4444	-0.0080	-0.0210	0.0183	0.3461
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1051	6.0608	-0.0232	-0.0607	0.0529	0.9689
ULS: 8. 0.6D + 0.7E	-0.0218	1.4666	-0.0048	-0.0126	0.0110	0.2077
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7946	6.4308	-0.0063	-0.0220	-0.0165	26.4048
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.7946	6.4308	-0.0063	-0.0220	-0.0165	26.4048
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2933	-0.9210	-0.0076	-0.0157	0.0419	-20.9126
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8819	-0.3459	-0.0222	-0.0504	0.0850	-24.8585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1739	9.0506	-0.0219	-0.0615	0.0268	20.5129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1739	9.0506	-0.0219	-0.0615	0.0268	20.5129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6420	3.5368	-0.0229	-0.0567	0.0706	-14.9752
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3335	3.9681	-0.0338	-0.0828	0.1030	-17.9346
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1050	5.4342	-0.0067	-0.0218	-0.0078	19.8901
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1050	5.4342	-0.0067	-0.0218	-0.0078	19.8901
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7109	-0.0797	-0.0077	-0.0170	0.0360	-15.5979
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4024	0.3517	-0.0186	-0.0431	0.0683	-18.5574
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7801	5.4530	-0.0031	-0.0137	-0.0239	26.2664
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.7801	5.4530	-0.0031	-0.0137	-0.0239	26.2664
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3078	-1.8988	-0.0044	-0.0073	0.0346	-21.0510
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8964	-1.3236	-0.0190	-0.0421	0.0777	-24.9970

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	13.9697
Shear X	-4.6853
Shear Z	-0.0555
Moment X	-0.1379
Moment Y (Twist)	0.1608
Moment Z	44.7840

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	9.0506
Shear X	-2.7946
Shear Z	-0.0338
Moment X	-0.0828
Moment Y (Twist)	0.1030
Moment Z	26.4048

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	-0.0363	2.4443	0.0080	0.0204	-0.0180	0.3459
ULS: 2. D + L	-0.0363	2.4443	0.0080	0.0204	-0.0180	0.3459
ULS: 3. D + (S or Lr or R)	-0.1280	7.2662	0.0282	0.0719	-0.0633	1.1757
ULS: 3. D + (S or Lr or R)	-0.0363	2.4443	0.0080	0.0204	-0.0180	0.3459
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1051	6.0607	0.0231	0.0590	-0.0520	0.9683
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0363	2.4443	0.0080	0.0204	-0.0180	0.3459
ULS: 5b. D + 0.7E	-0.0363	2.4443	0.0080	0.0204	-0.0180	0.3459
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1051	6.0607	0.0231	0.0590	-0.0520	0.9683
ULS: 8. 0.6D + 0.7E	-0.0218	1.4666	0.0048	0.0122	-0.0108	0.2075
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7945	6.4306	0.0061	0.0205	0.0162	26.4040
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.7945	6.4306	0.0061	0.0205	0.0162	26.4040
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2932	-0.9210	0.0077	0.0160	-0.0411	-20.9123
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.8819	-0.3459	0.0223	0.0506	-0.0841	-24.8584

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1738	9.0505	0.0217	0.0591	-0.0263	20.5118
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1738	9.0505	0.0217	0.0591	-0.0263	20.5118
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6421	3.5368	0.0229	0.0557	-0.0693	-14.9754
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3336	3.9681	0.0339	0.0817	-0.1016	-17.9350
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1049	5.4341	0.0066	0.0205	0.0077	19.8895
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.1049	5.4341	0.0066	0.0205	0.0077	19.8895
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7109	-0.0796	0.0078	0.0171	-0.0353	-15.5978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4024	0.3517	0.0187	0.0430	-0.0676	-18.5574
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7800	5.4529	0.0029	0.0123	0.0234	26.2656
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.7800	5.4529	0.0029	0.0123	0.0234	26.2656
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.3077	-1.8987	0.0045	0.0078	-0.0339	-21.0507
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.8964	-1.3236	0.0191	0.0424	-0.0769	-24.9968

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	13.9694
Shear X	-4.6851
Shear Z	0.0555
Moment X	0.1359
Moment Y (Twist)	0.1589
Moment Z	44.7825

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	9.0505
Shear X	-2.7945
Shear Z	0.0339
Moment X	0.0817
Moment Y (Twist)	0.1016
Moment Z	26.4040

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	0.0363	1.8520	-0.0950	-0.2409	0.0615	-0.2894
ULS: 2. D + L	0.0363	1.8520	-0.0950	-0.2409	0.0615	-0.2894
ULS: 3. D + (S or Lr or R)	0.1280	5.1762	-0.3365	-0.8550	0.2188	-1.0827
ULS: 3. D + (S or Lr or R)	0.0363	1.8520	-0.0950	-0.2409	0.0615	-0.2894
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1051	4.3452	-0.2762	-0.7014	0.1795	-0.8844
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0363	1.8520	-0.0950	-0.2409	0.0615	-0.2894
ULS: 5b. D + 0.7E	0.0363	1.8520	-0.0950	-0.2409	0.0615	-0.2894
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1051	4.3452	-0.2762	-0.7014	0.1795	-0.8844
ULS: 8. 0.6D + 0.7E	0.0218	1.1112	-0.0570	-0.1445	0.0369	-0.1736
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8779	4.5386	-0.3655	-0.8811	0.7714	18.1465
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8779	4.5386	-0.3655	-0.8811	0.7714	18.1465
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6491	-0.4129	0.1294	0.2899	-0.5278	-15.3921
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4034	-0.0497	0.1173	0.2605	-0.5050	-18.6509
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3305	6.3601	-0.4790	-1.1816	0.7119	12.9425
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3305	6.3601	-0.4790	-1.1816	0.7119	12.9425
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3147	2.6465	-0.1079	-0.3033	-0.2625	-12.2114
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1304	2.9189	-0.1169	-0.3254	-0.2454	-14.6555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3994	3.8669	-0.2979	-0.7210	0.5939	13.5375
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3994	3.8669	-0.2979	-0.7210	0.5939	13.5375
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2459	0.1533	0.0733	0.1572	-0.3805	-11.6164
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0616	0.4258	0.0642	0.1352	-0.3634	-14.0605

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8924	3.7978	-0.3275	-0.7847	0.7468	18.2622
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8924	3.7978	-0.3275	-0.7847	0.7468	18.2622
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6346	-1.1537	0.1674	0.3863	-0.5524	-15.2763
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3889	-0.7905	0.1553	0.3569	-0.5296	-18.5351

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.7806
Shear X	-3.1903
Shear Z	-0.7303
Moment X	-1.8214
Moment Y (Twist)	1.3420
Moment Z	31.6478

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.3601
Shear X	-1.8924
Shear Z	-0.4790
Moment X	-1.1816
Moment Y (Twist)	0.7714
Moment Z	18.6509

Project Details

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

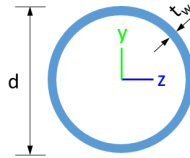


Design Input Information

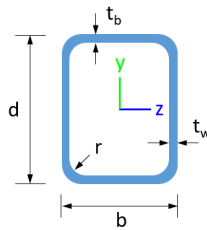
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

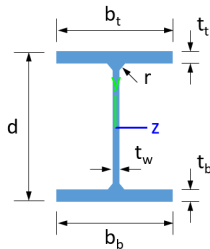
Section Dimensions



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



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------

315	20	10.2 0	10.2 0	10. 20	1.08,1.08,1.08,1.08,1.08,1.08,1.07,1.07,1.07,1.09,1.10,1.07,1.07,1.08,1.09,1.08,1.08,1.08,1.08,1.07,1.07,1.09, 9,1.10,1.07,1.07,1.08,1.09	30 0	20 0	1
316	20	10.2 0	10.2 0	10. 20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.11,1.08,1.08,1.08,1.59,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08, 8,1.12,1.08,1.08,1.08,1.17	30 0	20 0	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	237.07	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	229.64	27.16	27.16	75.30	75.30
13	159.30	97.43	38.78	6.46	56.26	44.91
14	159.30	97.43	46.90	6.46	56.26	44.91
15	159.30	137.23	46.90	6.46	56.26	44.91
16	159.30	137.23	46.90	6.46	56.26	44.91
101	377.97	237.07	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	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	31.85	6.46	56.26	44.91
114	159.30	97.43	32.28	6.46	56.26	44.91
115	159.30	32.87	20.99	6.46	56.26	44.91
116	159.30	32.87	21.20	6.46	56.26	44.91
201	377.97	237.07	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	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

212	251.01	240.00	27.10	27.10	75.30	75.30
213	159.30	97.43	31.84	6.46	56.26	44.91
214	159.30	97.43	32.29	6.46	56.26	44.91
215	159.30	32.87	21.48	6.46	56.26	44.91
216	159.30	32.87	21.75	6.46	56.26	44.91
301	377.97	237.07	83.29	83.29	113.39	113.39
302	251.01	229.64	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	137.23	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	137.23	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	38.79	6.46	56.26	44.91
314	159.30	97.43	46.90	6.46	56.26	44.91
315	159.30	32.87	20.68	6.46	56.26	44.91
316	159.30	32.87	20.72	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	0.041	0.380	0.060	0.028	0.006	0.401	#13	0.399	Not Required	Pass
2	0.002	0.184	0.088	0.046	0.019	0.245	#13	0.054	Not Required	Pass
3	0.005	0.315	0.020	0.030	0.005	0.320	#21	0.046	Not Required	Pass
4	0.004	0.310	0.060	0.031	0.014	0.372	#21	0.122	Not Required	Pass
5	0.004	0.195	0.023	0.031	0.006	0.198	#21	0.076	Not Required	Pass
6	0.010	0.482	0.107	0.049	0.030	0.593	#21	0.046	Not Required	Pass
7	0.010	0.298	0.163	0.048	0.041	0.337	#21	0.076	Not Required	Pass
8	0.004	0.109	0.171	0.027	0.019	0.203	#21	0.102	Not Required	Pass
9	0.005	0.044	0.083	0.003	0.005	0.113	#21	0.206	Not Required	Pass
10	0.011	0.457	0.148	0.046	0.032	0.530	#21	0.082	Not Required	Pass
11	0.007	0.106	0.179	0.029	0.019	0.210	#21	0.102	Not Required	Pass
12	0.001	0.367	0.138	0.080	0.025	0.468	#13	0.174	Not Required	Pass
13	0.011	0.051	0.450	0.039	0.025	0.456	#24	0.306	Not Required	Pass
14	0.004	0.055	0.439	0.037	0.025	0.453	#24	0.204	Not Required	Pass
15	0.000	0.003	0.011	0.005	0.003	0.014	#21	Not Required	Not Required	Pass
16	0.000	0.003	0.011	0.005	0.003	0.014	#21	Not Required	Not Required	Pass
101	0.059	0.538	0.005	0.041	0.000	0.564	#13	0.399	Not Required	Pass
102	0.003	0.409	0.174	0.091	0.031	0.540	#21	0.036	Not Required	Pass
103	0.010	0.583	0.060	0.058	0.008	0.648	#21	0.046	Not Required	Pass
104	0.010	0.587	0.162	0.059	0.035	0.697	#21	0.082	Not Required	Pass
105	0.010	0.361	0.171	0.057	0.044	0.406	#21	0.076	Not Required	Pass
106	0.010	0.580	0.058	0.058	0.008	0.639	#21	0.046	Not Required	Pass
107	0.010	0.360	0.163	0.057	0.042	0.403	#21	0.076	Not Required	Pass
108	0.005	0.035	0.166	0.033	0.019	0.203	#21	0.102	Not Required	Pass
109	0.016	0.047	0.044	0.001	0.000	0.095	#21	0.206	Not Required	Pass
110	0.009	0.572	0.158	0.057	0.034	0.683	#21	0.082	Not Required	Pass

111	0.007	0.050	0.172	0.033	0.019	0.198	#21	0.102	Not Required	Pass
112	0.003	0.400	0.174	0.090	0.032	0.531	#13	0.036	Not Required	Pass
113	0.011	0.163	0.463	0.046	0.025	0.598	#21	0.306	Not Required	Pass
114	0.007	0.187	0.457	0.047	0.025	0.614	#21	0.306	Not Required	Pass
115	0.032	0.305	0.239	0.036	0.020	0.550	#21	0.780	Not Required	Pass
116	0.009	0.289	0.235	0.038	0.020	0.521	#21	0.780	Not Required	Pass
201	0.059	0.538	0.005	0.041	0.000	0.564	#13	0.399	Not Required	Pass
202	0.003	0.400	0.174	0.090	0.032	0.531	#13	0.036	Not Required	Pass
203	0.010	0.580	0.058	0.058	0.008	0.639	#21	0.046	Not Required	Pass
204	0.009	0.572	0.158	0.057	0.034	0.683	#21	0.082	Not Required	Pass
205	0.010	0.360	0.163	0.057	0.042	0.403	#21	0.076	Not Required	Pass
206	0.010	0.583	0.060	0.058	0.008	0.648	#21	0.046	Not Required	Pass
207	0.010	0.361	0.171	0.057	0.044	0.406	#21	0.076	Not Required	Pass
208	0.004	0.059	0.185	0.038	0.020	0.205	#21	0.102	Not Required	Pass
209	0.016	0.047	0.044	0.001	0.000	0.095	#21	0.206	Not Required	Pass
210	0.010	0.587	0.162	0.059	0.035	0.697	#21	0.082	Not Required	Pass
211	0.007	0.076	0.190	0.036	0.020	0.198	#21	0.102	Not Required	Pass
212	0.003	0.409	0.174	0.091	0.031	0.540	#21	0.036	Not Required	Pass
213	0.011	0.163	0.463	0.046	0.025	0.598	#21	0.306	Not Required	Pass
214	0.007	0.187	0.457	0.047	0.025	0.614	#21	0.306	Not Required	Pass
215	0.030	0.216	0.244	0.033	0.019	0.464	#21	0.780	Not Required	Pass
216	0.011	0.171	0.241	0.033	0.019	0.413	#21	0.780	Not Required	Pass
301	0.041	0.380	0.060	0.028	0.006	0.401	#13	0.399	Not Required	Pass
302	0.001	0.367	0.138	0.080	0.025	0.468	#13	0.174	Not Required	Pass
303	0.010	0.482	0.107	0.049	0.030	0.593	#21	0.046	Not Required	Pass
304	0.011	0.457	0.148	0.046	0.032	0.530	#21	0.082	Not Required	Pass
305	0.010	0.298	0.163	0.047	0.041	0.337	#21	0.076	Not Required	Pass
306	0.005	0.315	0.019	0.030	0.005	0.320	#21	0.046	Not Required	Pass
307	0.004	0.195	0.023	0.031	0.006	0.198	#21	0.076	Not Required	Pass
308	0.000	0.003	0.011	0.005	0.003	0.014	#21	Not Required	Not Required	Pass
309	0.005	0.044	0.083	0.003	0.005	0.113	#21	0.206	Not Required	Pass
310	0.004	0.310	0.060	0.031	0.014	0.372	#21	0.122	Not Required	Pass
311	0.000	0.003	0.011	0.005	0.003	0.014	#21	Not Required	Not Required	Pass
312	0.002	0.184	0.088	0.046	0.019	0.245	#13	0.054	Not Required	Pass
313	0.011	0.051	0.450	0.039	0.025	0.456	#24	0.204	Not Required	Pass
314	0.004	0.055	0.439	0.037	0.025	0.453	#24	0.306	Not Required	Pass
315	0.032	0.324	0.243	0.029	0.019	0.578	#21	0.780	Not Required	Pass
316	0.009	0.316	0.240	0.027	0.019	0.549	#21	0.780	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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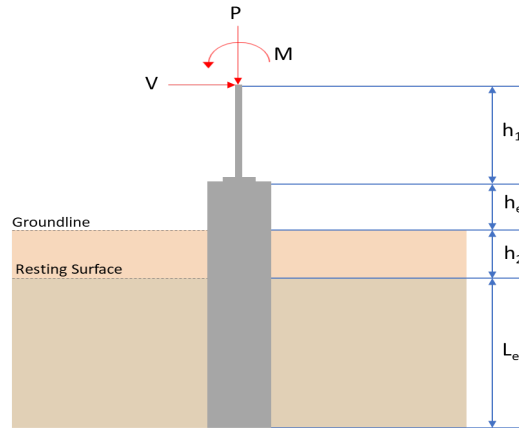
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.360	9.781
V_x (kip)	-1.892	-3.190
V_z (kip)	0.479	0.731
M_x (kipft)	1.180	1.818
M_z (kipft)	18.650	31.647

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,

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

$$H_o = \frac{(-1.892 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.63067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.65 \text{ kipft}) + ((-1.892 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.2167 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.1058 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.479 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.15967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.18 \text{ kipft}) + ((0.479 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.39333 \text{ kipft/ft}$$

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:

$L_{e,z} = 4.9948 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.1058 \text{ ft}), (4.9948 \text{ ft})]$$

$$L_{e,req} = 7.106 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.106 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.88825$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.36 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.89976 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.89976 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.44988$$

Status: **PASS**
Ratio: **0.450**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.6667$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.63067 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.2167 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2167 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (6.2167 \text{ kipft/ft})) + (4 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5674 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.2167 \text{ kipft/ft})) + (3 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (6.2167 \text{ kipft/ft})) + (2 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.20362 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.2167 \text{ kipft/ft})) + ((-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.088 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5674 \text{ ft})}{2}$$

$$p_a = 0.41755 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20362 \text{ kip/ft}^2)}{(0.41755 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.48764$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.088 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90667$$

Status: **PASS**
Ratio: **0.490**

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

Considering z-direction:

$H_o = 0.15967 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.39333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.39333 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.15967 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.39333 \text{ kipft/ft})) + (4 \times (0.15967 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7894 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.39333 \text{ kipft/ft})) + (3 \times (0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.39333 \text{ kipft/ft})) + (2 \times (0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.144 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.39333 \text{ kipft/ft})) + ((0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.30396 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7894 \text{ ft})}{2}$$

$$p_a = 0.4342 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.144 \text{ kip/ft}^2)}{(0.4342 \text{ kip/ft}^2)}$$

$$(0.30396 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.33164$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

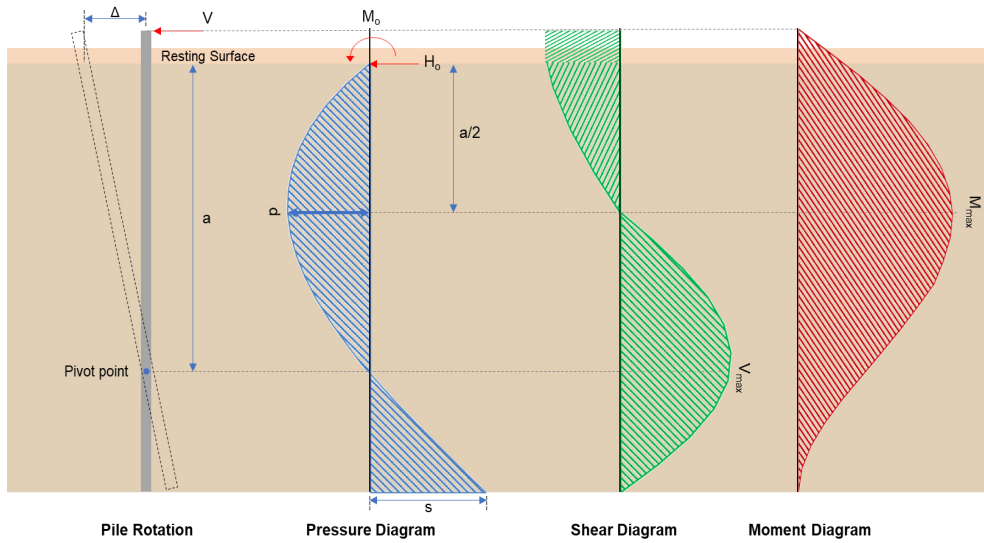
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.30396 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.2533$$

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

Status: **PASS**
Ratio: **0.250**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.19 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.0633 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.647 \text{ kipft}) + ((-3.19 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.549 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.549 \text{ kipft/ft})}{(-1.0633 \text{ kip/ft})}$$

$$E = 9.9207 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.549 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.0633 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (10.549 \text{ kipft/ft})) + (4 \times (-1.0633 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5664 \text{ ft}$$

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

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

$$V_{max} = ((-1.0633 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.9207 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5664 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.9207 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5664 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.1041 \text{ kip}$$

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

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

$$M_{max} = ((-1.0633 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(9.9207 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.9207 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.9207 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.11 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.731 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.24367 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.818 \text{ kipft}) + ((0.731 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.606 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.606 \text{ kipft/ft})}{(0.24367 \text{ kip/ft})}$$

$$E = 2.487 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.606 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.24367 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.606 \text{ kipft/ft})) + (4 \times (0.24367 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.788 \text{ ft}$$

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

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

$$[\setminus L_e \quad / \setminus L_e /]]$$

$$V_{max} = ((0.24367 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.487 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.788 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.487 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.788 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.89274 \text{ kip}$$

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

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

$$M_{max} = ((0.24367 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.487 \text{ ft})}{(8 \text{ ft})} + \frac{(5.788 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.487 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.788 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.487 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.788 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.0524 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.781 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.068 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.068 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.781 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0078004$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.781 \text{ kip} \rightarrow 9781 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9781 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.098 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.098 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.098 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.098 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.275 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.1041 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.1041 \text{ kip})}{(74.275 \text{ kip})}$$

$$Ratio = 0.12257$$

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

Considering z-direction:

$V_{max} = 0.89274 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.89274 \text{ kip})}{(74.275 \text{ kip})}$$

$$Ratio = 0.012019$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.11 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.11 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.54992$$

Status: **PASS**
Ratio: **0.550**

Considering z-direction:

$M_{max} = 3.0524 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(3.0524 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.049211$$

Status: **PASS**
Ratio: **0.050**

REFERENCES	CALCULATIONS	RESULTS
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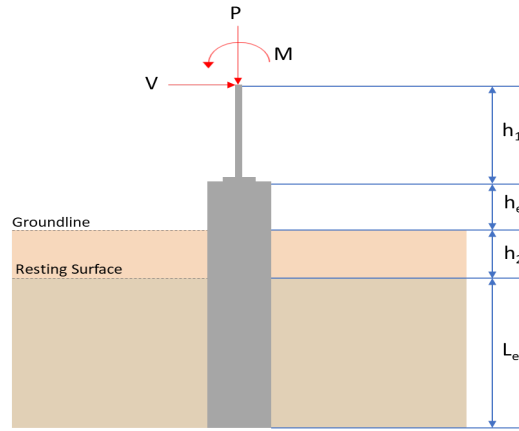
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.360	9.781
V_x (kip)	-1.892	-3.190
V_z (kip)	-0.479	-0.730
M_x (kipft)	-1.182	-1.821
M_z (kipft)	18.651	31.648

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,

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

$$H_o = \frac{(-1.892 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.63067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.651 \text{ kipft}) + ((-1.892 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.217 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.106 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.479 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.15967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.182 \text{ kipft}) + ((-0.479 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.394 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.3865 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.106 \text{ ft}), (2.3865 \text{ ft})]$$

$$L_{e,req} = 7.106 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.106 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.88825$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(6.36 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.89976 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.89976 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.44988$$

Status: **PASS**
Ratio: **0.450**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.6667$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.63067 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.217 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.217 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (6.217 \text{ kipft/ft})) + (4 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5674 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.217 \text{ kipft/ft})) + (3 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (6.217 \text{ kipft/ft})) + (2 \times (-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.20365 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.217 \text{ kipft/ft})) + ((-0.63067 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.0881 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5674 \text{ ft})}{2}$$

$$p_a = 0.41755 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20365 \text{ kip/ft}^2)}{(0.41755 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.48772$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0881 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90675$$

Status: **PASS**
Ratio: **0.490**

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

Considering z-direction:

$H_o = -0.15967 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.394 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.394 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.15967 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.394 \text{ kipft/ft})) + (4 \times (-0.15967 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7891 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.394 \text{ kipft/ft})) + (3 \times (-0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.394 \text{ kipft/ft})) + (2 \times (-0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.068246 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.394 \text{ kipft/ft})) + ((-0.15967 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = -0.072062 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7891 \text{ ft})}{2}$$

$$p_a = 0.43418 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.068246 \text{ kip/ft}^2)}{(0.43418 \text{ kip/ft}^2)}$$

$$(-0.072062 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.15718$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

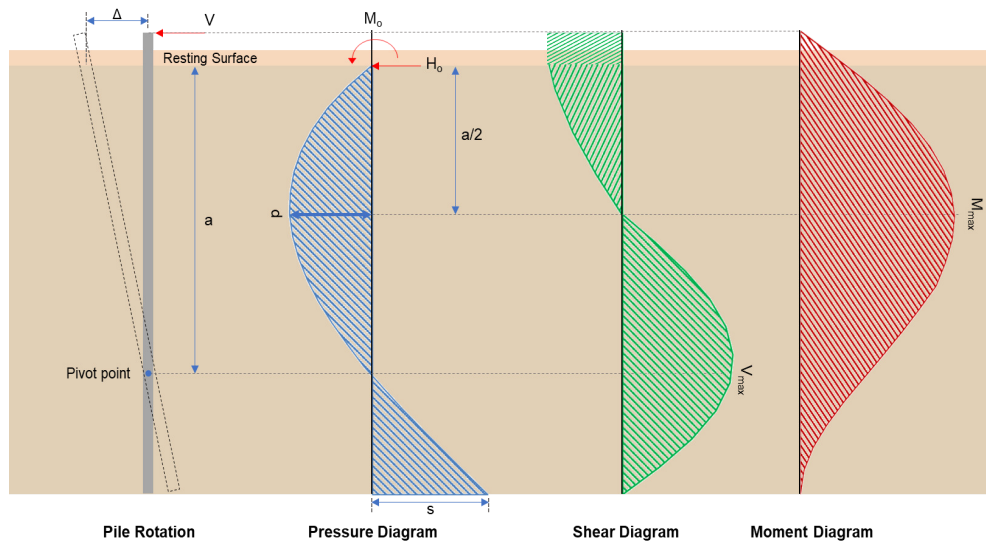
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.072062 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.060052$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.19 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.0633 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.648 \text{ kipft}) + ((-3.19 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.549 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.549 \text{ kipft/ft})}{(-1.0633 \text{ kip/ft})}$$

$$E = 9.921 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.549 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.0633 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (10.549 \text{ kipft/ft})) + (4 \times (-1.0633 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5664 \text{ ft}$$

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

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

$$V_{max} = ((-1.0633 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.921 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5664 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.921 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5664 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.1043 \text{ kip}$$

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

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

$$M_{max} = ((-1.0633 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(9.921 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.921 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.921 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5664 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.111 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.73 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.24333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.821 \text{ kipft}) + ((-0.73 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.607 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.607 \text{ kipft/ft})}{(-0.24333 \text{ kip/ft})}$$

$$E = 2.4945 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.607 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.24333 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.607 \text{ kipft/ft})) + (4 \times (-0.24333 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7876 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.24333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4945 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7876 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.4945 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7876 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.89271 \text{ kip}$$

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

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

$$M_{max} = ((-0.24333 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.4945 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7876 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.4945 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7876 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4945 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7876 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.053 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

Table 22.4.2.1

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.781 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.068 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.068 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$(1.8322 \text{ in}^2)$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.781 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0078004$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.781 \text{ kip} \rightarrow 9781 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(9781 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.098 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.098 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.098 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.098 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.275 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.1043 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.1043 \text{ kip})}{(74.275 \text{ kip})}$$

$$Ratio = 0.12258$$

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

Considering z-direction:

$V_{max} = 0.89271 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.89271 \text{ kip})}{(74.275 \text{ kip})}$$

$$Ratio = 0.012019$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.111 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.111 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.54993$$

Status: **PASS**
Ratio: **0.550**

Considering z-direction:

$M_{max} = 3.053 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(3.053 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.049221$$

Status: **PASS**
Ratio: **0.050**

REFERENCES	CALCULATIONS	RESULTS
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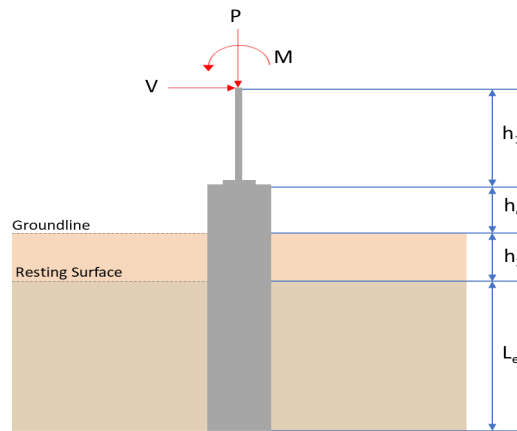
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.5$ 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)	9.051	13.970
V_x (kip)	-2.795	-4.685
V_z (kip)	-0.034	-0.055
M_x (kipft)	-0.083	-0.138
M_z (kipft)	26.405	44.784

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,

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

$$H_o = \frac{(-2.795 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.93167 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.405 \text{ kipft}) + ((-2.795 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.8017 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5986 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.034 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.011333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.083 \text{ kipft}) + ((-0.034 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.027667 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2822 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5986 \text{ ft}), (1.2822 \text{ ft})]$$

$$L_{e,req} = 7.599 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(7.599 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.894$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.051 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2805 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.2805 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.64023$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.93167 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8017 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8017 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (8.8017 \text{ kipft/ft})) + (4 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9322 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (8.8017 \text{ kipft/ft})) + (3 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (8.8017 \text{ kipft/ft})) + (2 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.20226 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (8.8017 \text{ kipft/ft})) + ((-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2633 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9322 \text{ ft})}{2}$$

$$p_a = 0.44492 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20226 \text{ kip/ft}^2)}{(0.44492 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.45461$$

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

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

$$\text{Ratio} = \frac{(1.2633 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99082$$

Status: **PASS**
Ratio: **0.450**

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = -0.011333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.027667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.027667 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.011333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.027667 \text{ kipft/ft})) + (4 \times (-0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1617 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.027667 \text{ kipft/ft})) + (3 \times (-0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.027667 \text{ kipft/ft})) + (2 \times (-0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.0047282 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.027667 \text{ kipft/ft})) + ((-0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.0053484 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1617 \text{ ft})}{2}$$

$$p_a = 0.46213 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0047282 \text{ kip/ft}^2)}{(0.46213 \text{ kip/ft}^2)}$$

$$(0.70219 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.010231$$

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

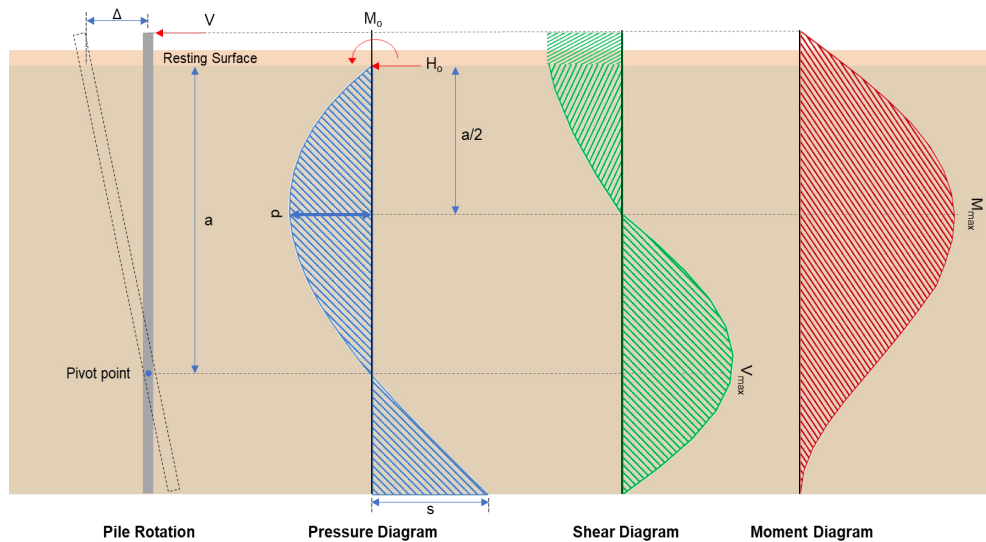
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0053484 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0041949$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.685 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.5617 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.784 \text{ kipft}) + ((-4.685 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.928 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.928 \text{ kipft/ft})}{(-1.5617 \text{ kip/ft})}$$

$$E = 9.559 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.928 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.5617 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (14.928 \text{ kipft/ft})) + (4 \times (-1.5617 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9303 \text{ ft}$$

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

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

$$V_{max} = ((-1.5617 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.559 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.9303 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.559 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.9303 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.415 \text{ kip}$$

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

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

$$M_{max} = ((-1.5617 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(9.559 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.559 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.559 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 49.169 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.055 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.018333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.138 \text{ kipft}) + ((-0.055 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.046 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.046 \text{ kipft/ft})}{(-0.018333 \text{ kip/ft})}$$

$$E = 2.5091 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.046 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.018333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.046 \text{ kipft/ft})) + (4 \times (-0.018333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1576 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right] / \left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.018333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5091 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.5091 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.065671 \text{ kip}$$

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

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

$$M_{max} = ((-0.018333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(2.5091 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.1576 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.5091 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5091 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.1576 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.23767 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.97 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.937 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.937 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</p> <p>s_{rebar} - Minimum spacing of reinforcement,</p> <p style="text-align: center;">$s_{rebar} = Max [1.5, (1.5 d_{bar})]$</p> <p style="text-align: center;">$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>s_{ties} - Maximum center-to-center spacing of ties,</p> <p style="text-align: center;">$s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), D]$</p> <p style="text-align: center;">$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio</i> - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.97 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.011141$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.97 \text{ kip} \rightarrow 13970 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13970 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.81 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.81 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.81 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.81 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.737 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.415 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(12.415 \text{ kip})}{(74.737 \text{ kip})}$$

$$Ratio = 0.16611$$

Status: **PASS**
Ratio: **0.170**

Considering z-direction:

$V_{max} = 0.065671 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.065671 \text{ kip})}{(74.737 \text{ kip})}$$

$$Ratio = 0.0008787$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 49.169 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(49.169 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.7927$$

Status: **PASS**
Ratio: **0.790**

Considering z-direction:

$M_{max} = 0.23767 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.23767 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0038318$$

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

REFERENCES	CALCULATIONS	RESULTS
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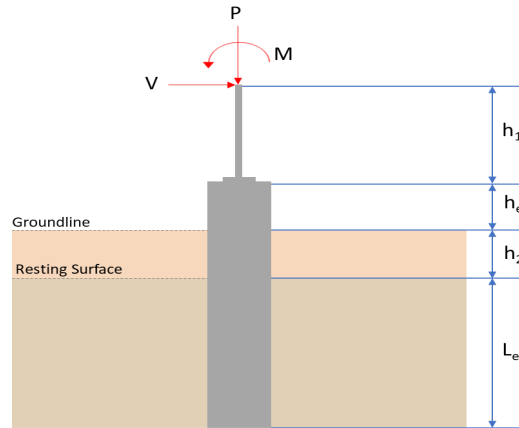
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.5$ 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)	9.050	13.969
V_x (kip)	-2.795	-4.685
V_z (kip)	0.034	0.055
M_x (kipft)	0.082	0.136
M_z (kipft)	26.404	44.782

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,

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

$$H_o = \frac{(-2.795 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.93167 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.404 \text{ kipft}) + ((-2.795 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.8013 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5984 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.034 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.011333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.082 \text{ kipft}) + ((0.034 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.027333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7431 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5984 \text{ ft}), (1.7431 \text{ ft})]$$

$$L_{e,req} = 7.598 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(7.598 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.89388$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.05 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2803 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.2803 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.64016$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.93167 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.8013 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.8013 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (8.8013 \text{ kipft/ft})) + (4 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9322 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (8.8013 \text{ kipft/ft})) + (3 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (8.8013 \text{ kipft/ft})) + (2 \times (-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.20224 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (8.8013 \text{ kipft/ft})) + ((-0.93167 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2632 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9322 \text{ ft})}{2}$$

$$p_a = 0.44492 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20224 \text{ kip/ft}^2)}{(0.44492 \text{ kip/ft}^2)}$$

$$Ratio = 0.45454$$

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

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

$$Ratio = \frac{(1.2632 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.99075$$

Status: **PASS**
Ratio: **0.450**

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = 0.011333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.027333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.027333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.011333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.027333 \text{ kipft/ft})) + (4 \times (0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1635 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.027333 \text{ kipft/ft})) + (3 \times (0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.027333 \text{ kipft/ft})) + (2 \times (0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.0094188 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.027333 \text{ kipft/ft})) + ((0.011333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.019698 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1635 \text{ ft})}{2}$$

$$p_a = 0.46226 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0094188 \text{ kip/ft}^2)}{(0.46226 \text{ kip/ft}^2)}$$

$$(0.02229 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.020375$$

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

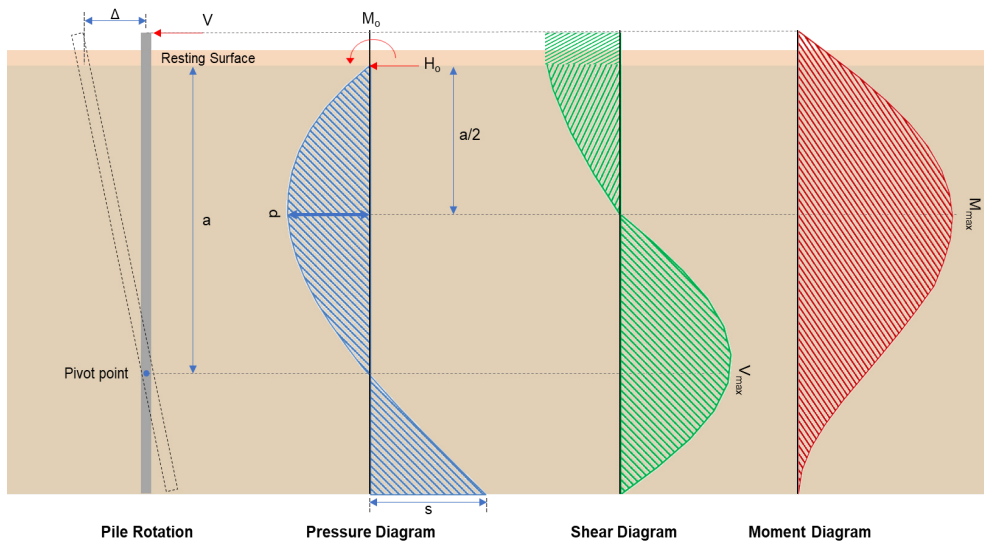
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.019698 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.015449$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.685 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.5617 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(44.782 \text{ kipft}) + ((-4.685 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.927 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.927 \text{ kipft/ft})}{(-1.5617 \text{ kip/ft})}$$

$$E = 9.5586 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.927 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.5617 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (14.927 \text{ kipft/ft})) + (4 \times (-1.5617 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9303 \text{ ft}$$

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

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

$$V_{max} = ((-1.5617 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.5586 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.9303 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.5586 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.9303 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.414 \text{ kip}$$

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

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

$$M_{max} = ((-1.5617 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(9.5586 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.5586 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.5586 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.9303 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 49.167 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.055 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.018333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.136 \text{ kipft}) + ((0.055 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.045333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.045333 \text{ kipft/ft})}{(0.018333 \text{ kip/ft})}$$

$$E = 2.4727 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.045333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.018333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.045333 \text{ kipft/ft})) + (4 \times (0.018333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1598 \text{ ft}$$

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

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

$$[\setminus L_e / \setminus L_e /]]$$

$$V_{max} = ((0.018333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4727 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.1598 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.4727 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.1598 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.065263 \text{ kip}$$

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

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

$$M_{max} = ((0.018333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(2.4727 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.1598 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.4727 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.1598 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4727 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.1598 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.23594 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.969 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.937 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.937 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{1.8322 \text{ in}^2}{1.8408 \text{ in}^2}$$

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;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.969 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.01114$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.969 \text{ kip} \rightarrow 13969 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13969 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.809 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.809 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.809 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.809 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.737 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.414 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.414 \text{ kip})}{(74.737 \text{ kip})}$$

$$Ratio = 0.16611$$

Status: **PASS**
Ratio: **0.170**

Considering z-direction:

$V_{max} = 0.065263 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.065263 \text{ kip})}{(74.737 \text{ kip})}$$

$$Ratio = 0.00087324$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 49.167 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(49.167 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.79267$$

Status: **PASS**
Ratio: **0.790**

Considering z-direction:

$M_{max} = 0.23594 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.23594 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0038039$$

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