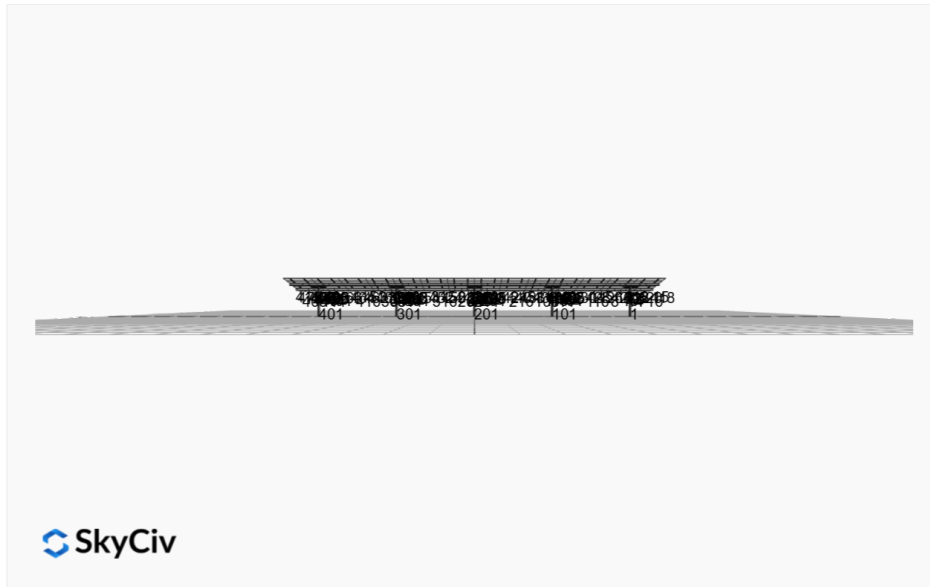


Project Name: MTSOLAR_A262L2JE645HA **Date:** Thu Jun 05 2025
Location: 140 N Main St, Hillsboro, IL 62049, USA **Number of Modules:** 70
Unique ID: 5P-22.5-6TOP-HD-45-L-5Hx14W-103G **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	105.80 ft
Winter Tilt Angle	10
Front Edge Clearance	7 ft

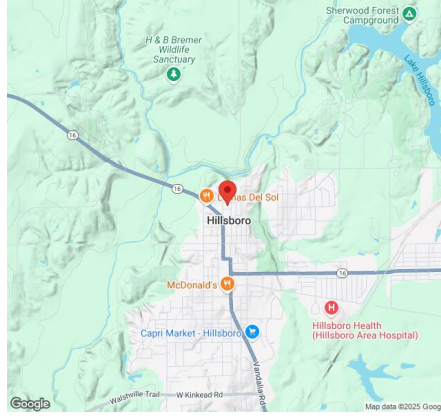
MT Solar Bill of Materials (5P-22.5-6TOP-HD-45-L-5Hx14W-103G)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	16
MTS-CLAMP-ANGLE-4PK	Angle Clamp	14

Rail Bill of Materials

Part	Qty
Rails (226in)	28
Rail Attachment	112
Module Mid Clamp	112
Module End Clamp	56
Ground Lug	14

Site Details:



Site Address: 140 N Main St, Hillsboro, IL 62049, USA

Array Specification

Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	5
Number of Columns:	14
Total Number of Modules:	70
Winter Tilt Angle:	10
Front Edge Clearance:	7
Total Array Height at Tilt:	10.27 ft
Total Frame Length:	105.00 ft
Module Info/Notes:	Znshine 550
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	105.80 ft
Rail Length:	225.73 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.63 ft
Number of Poles:	5
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 7.00 ft Pile 2: 7.25 ft Pile 3: 7.25 ft Pile 4: 7.25 ft Pile 5: 7.00 ft
Foundation Volume:	9.359 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	140 N Main St, Hillsboro, IL 62049, USA
Wind Speed:	101 mph

Snow Load:

20 psf

Design Disclaimer

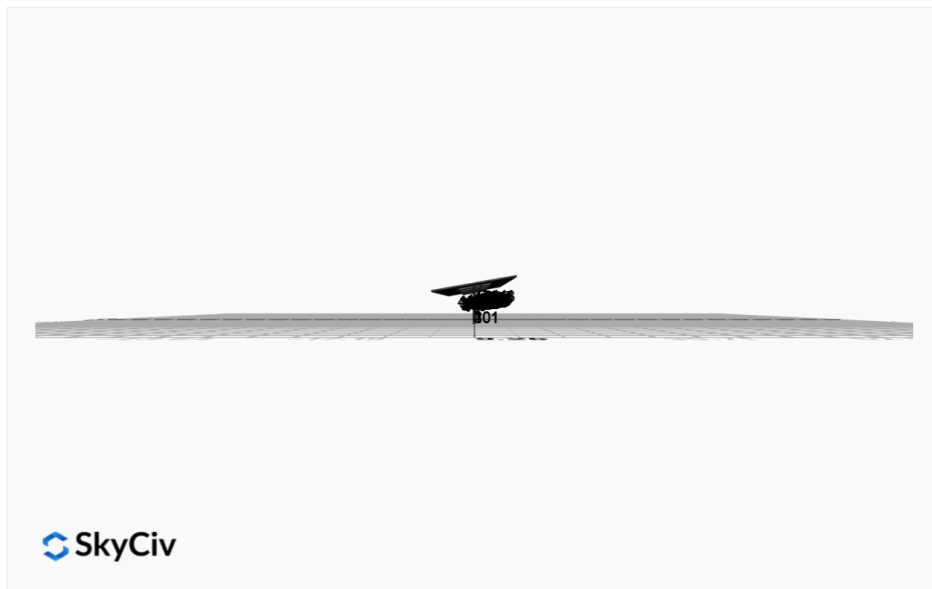
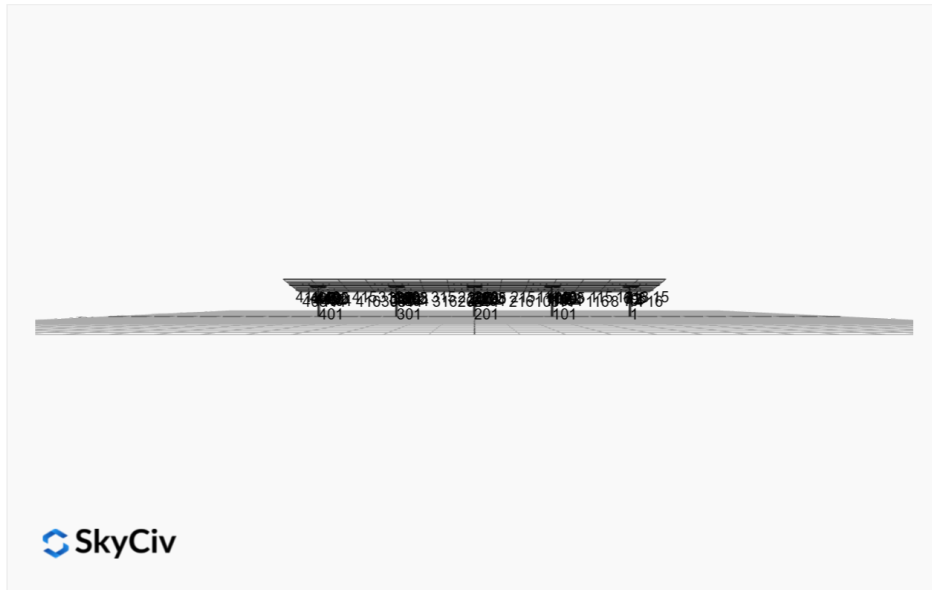
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

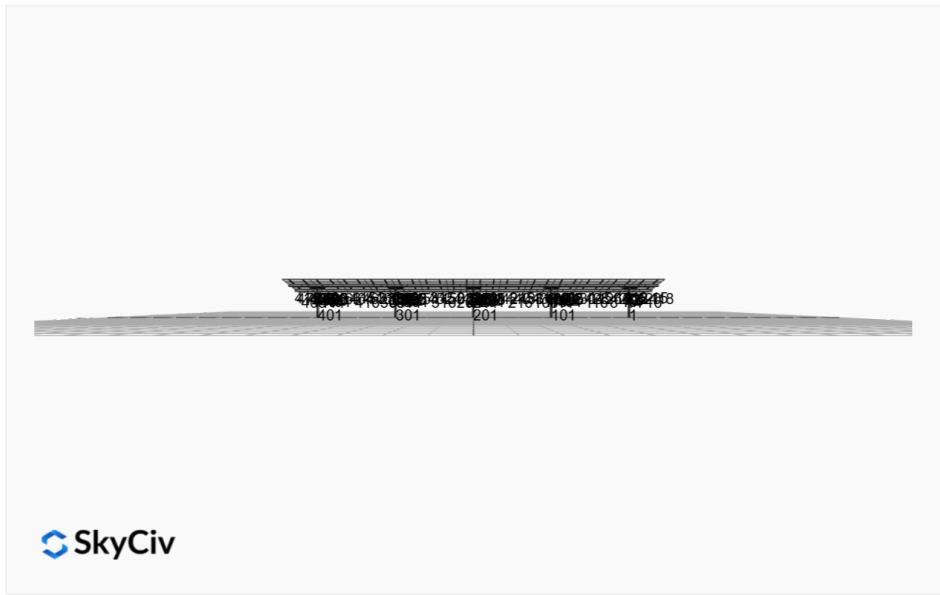
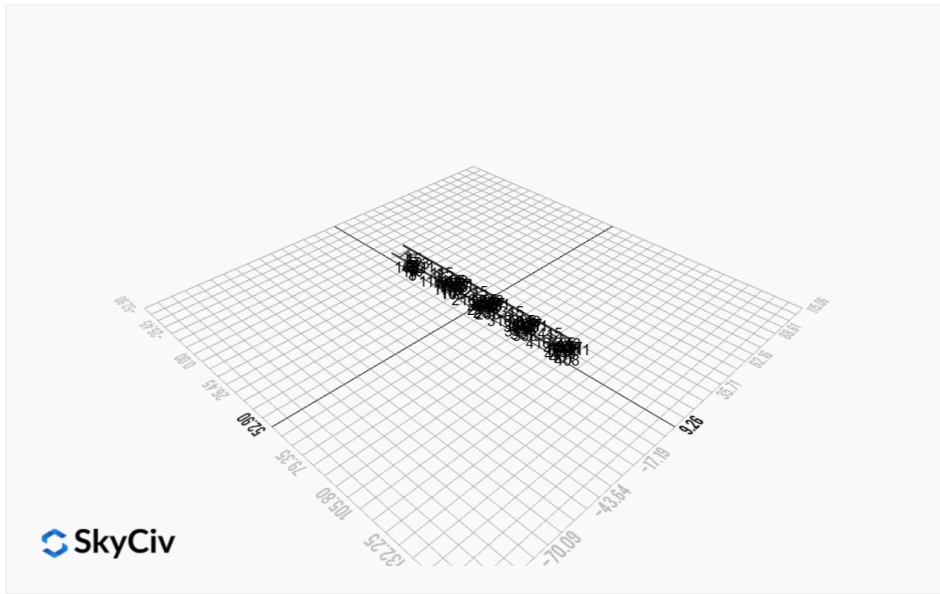
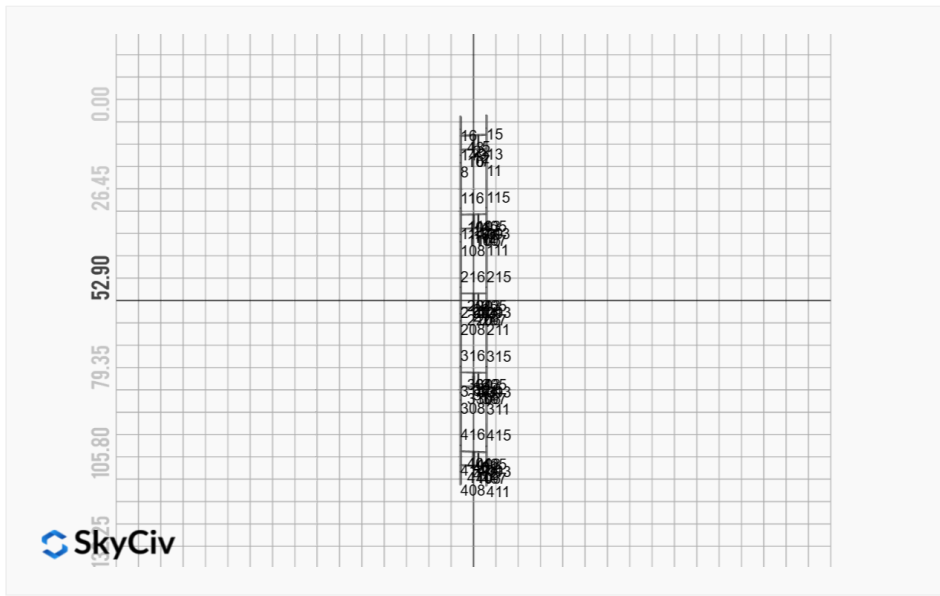
AutoDesigner Input

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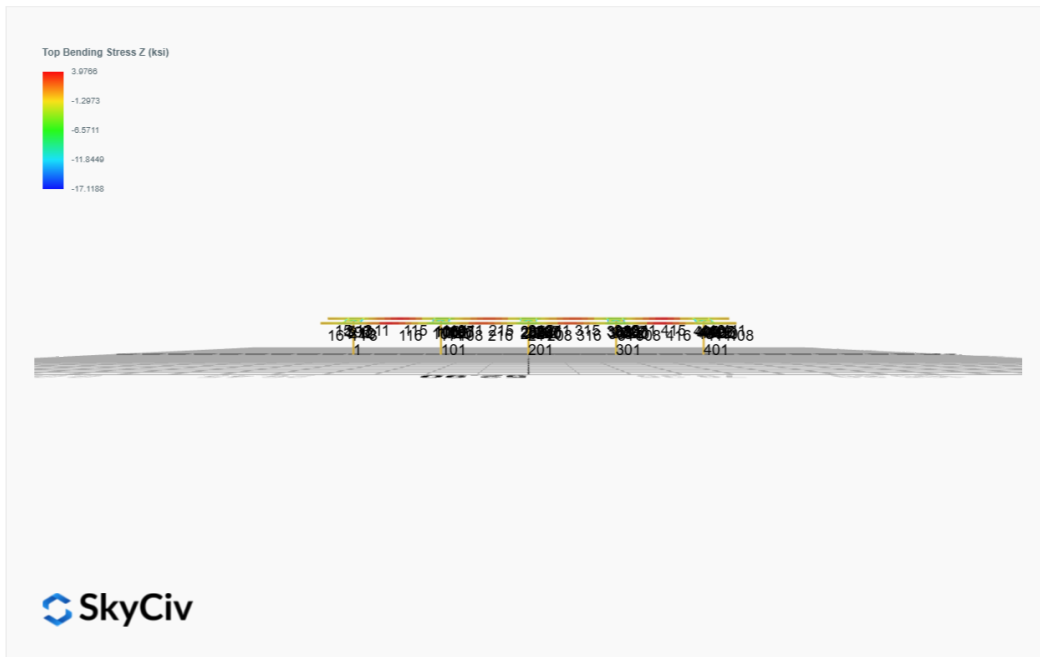
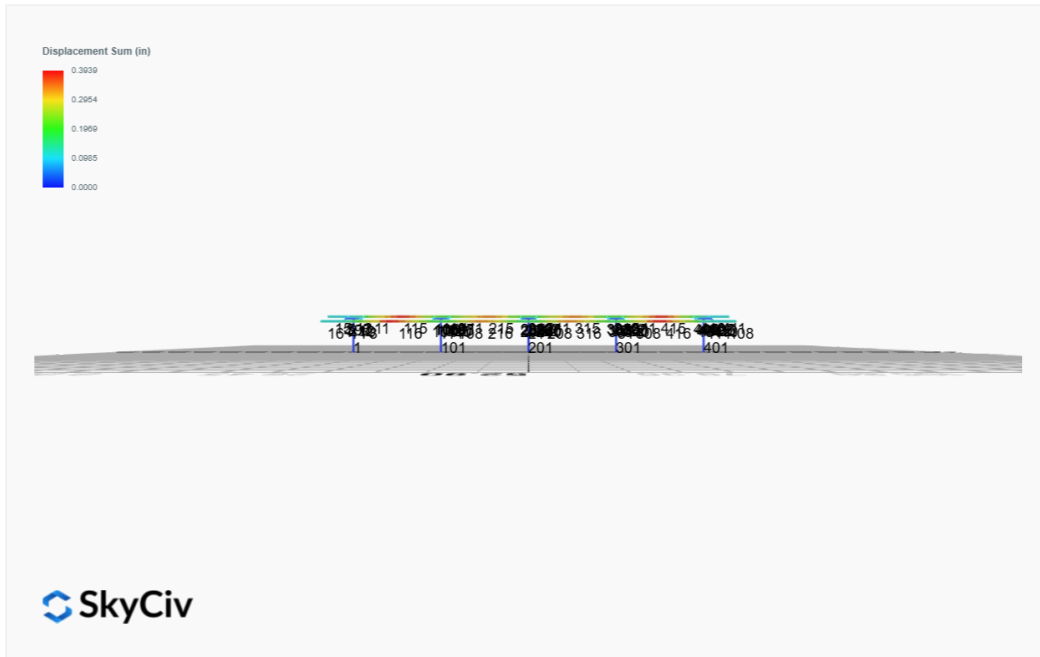
Design Notes:

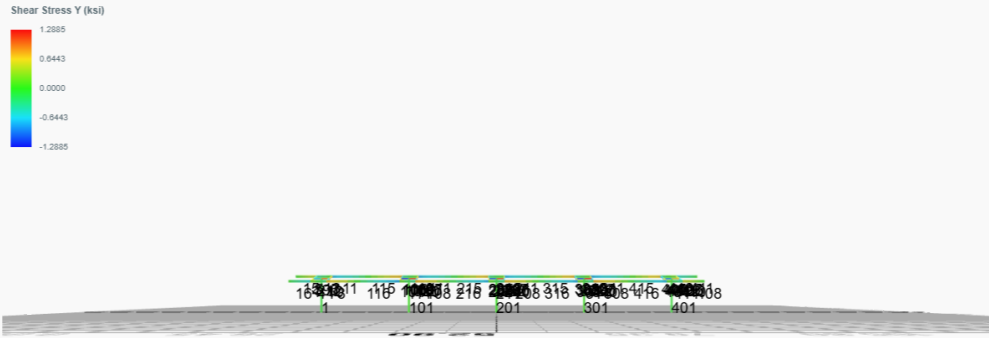
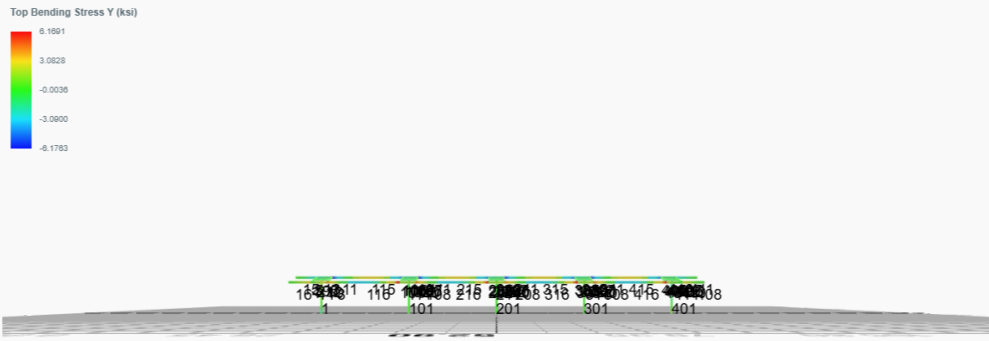
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

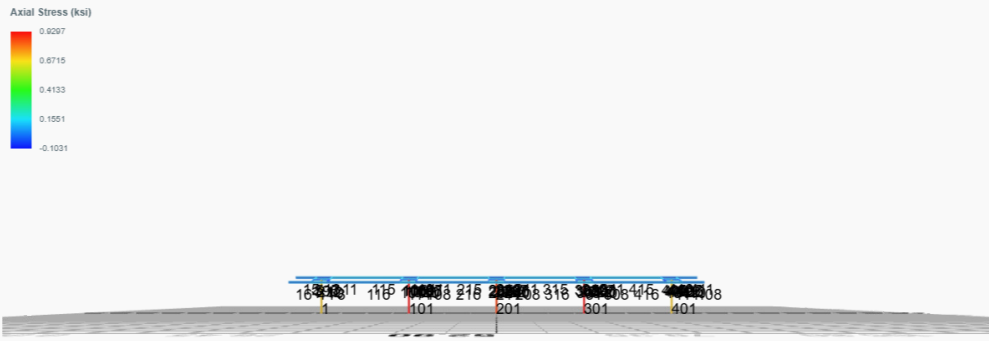




FEM Results (Envelope Worst Case for each member)







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	0.0093	2.2673	0.0922	0.2327	-0.0189	-0.0436
ULS: 2. D + L	0.0093	2.2673	0.0922	0.2327	-0.0189	-0.0436
ULS: 3. D + (S or Lr or R)	0.0293	6.3441	0.2917	0.7367	-0.0597	-0.1945
ULS: 3. D + (S or Lr or R)	0.0093	2.2673	0.0922	0.2327	-0.0189	-0.0436
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0243	5.3249	0.2418	0.6107	-0.0495	-0.1568
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0093	2.2673	0.0922	0.2327	-0.0189	-0.0436
ULS: 5b. D + 0.7E	0.0093	2.2673	0.0922	0.2327	-0.0189	-0.0436
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0243	5.3249	0.2418	0.6107	-0.0495	-0.1568
ULS: 8. 0.6D + 0.7E	0.0056	1.3604	0.0553	0.1396	-0.0113	-0.0262
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.4888	5.0384	0.2354	0.5897	-0.0900	6.5663
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.4888	5.0384	0.2354	0.5897	-0.0900	6.5663
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3525	0.3259	-0.0030	-0.0045	0.0251	-1.3394
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3222	0.5730	-0.0016	-0.0001	0.0329	-9.2253
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3493	7.4032	0.3492	0.8785	-0.1029	4.8006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3493	7.4032	0.3492	0.8785	-0.1029	4.8006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2817	3.8688	0.1704	0.4328	-0.0166	-1.1286
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2590	4.0541	0.1714	0.4361	-0.0107	-7.0431
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3643	4.3457	0.1996	0.5005	-0.0722	4.9138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3643	4.3457	0.1996	0.5005	-0.0722	4.9138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2667	0.8112	0.0208	0.0548	0.0141	-1.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2440	0.9965	0.0218	0.0581	0.0200	-6.9299
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.4925	4.1315	0.1985	0.4967	-0.0825	6.5837
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.4925	4.1315	0.1985	0.4967	-0.0825	6.5837
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3488	-0.5811	-0.0399	-0.0976	0.0326	-1.3219
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3185	-0.3340	-0.0385	-0.0932	0.0405	-9.2079

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	11.5526
Shear X	-0.8301
Shear Z	0.5522
Moment X	1.3948
Moment Y (Twist)	0.1627
Moment Z	15.8891

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	7.4032
Shear X	-0.4925
Shear Z	0.3492
Moment X	0.8785
Moment Y (Twist)	0.1029
Moment Z	9.2253

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	-0.0107	2.7830	-0.0148	-0.0388	0.0067	0.1085
ULS: 2. D + L	-0.0107	2.7830	-0.0148	-0.0388	0.0067	0.1085
ULS: 3. D + (S or Lr or R)	-0.0338	7.9721	-0.0468	-0.1230	0.0213	0.2914
ULS: 3. D + (S or Lr or R)	-0.0107	2.7830	-0.0148	-0.0388	0.0067	0.1085
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0281	6.6749	-0.0388	-0.1020	0.0177	0.2457

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0107	2.7830	-0.0148	-0.0388	0.0067	0.1085
ULS: 5b. D + 0.7E	-0.0107	2.7830	-0.0148	-0.0388	0.0067	0.1085
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0281	6.6749	-0.0388	-0.1020	0.0177	0.2457
ULS: 8. 0.6D + 0.7E	-0.0064	1.6698	-0.0089	-0.0233	0.0040	0.0651
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6234	6.3097	-0.0340	-0.0903	0.0089	8.1178
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6234	6.3097	-0.0340	-0.0903	0.0089	8.1178
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4246	0.3066	0.0009	0.0024	0.0005	-1.5689
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3549	0.6353	-0.0064	-0.0152	0.0124	-10.7442
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4876	9.3198	-0.0532	-0.1406	0.0193	6.2527
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4876	9.3198	-0.0532	-0.1406	0.0193	6.2527
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2984	4.8175	-0.0270	-0.0711	0.0130	-1.0123
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2462	5.0640	-0.0325	-0.0842	0.0220	-7.8938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4702	5.4280	-0.0292	-0.0774	0.0083	6.1155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4702	5.4280	-0.0292	-0.0774	0.0083	6.1155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3158	0.9257	-0.0030	-0.0079	0.0020	-1.1496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2635	1.1722	-0.0085	-0.0211	0.0110	-8.0310
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6191	5.1964	-0.0281	-0.0748	0.0062	8.0744
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6191	5.1964	-0.0281	-0.0748	0.0062	8.0744
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4289	-0.8067	0.0069	0.0179	-0.0022	-1.6123
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3592	-0.4779	-0.0005	0.0004	0.0097	-10.7876

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	14.5812
Shear X	-1.0448
Shear Z	-0.0853
Moment X	-0.2258
Moment Y (Twist)	0.0375
Moment Z	18.4969

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	9.3198
Shear X	-0.6234
Shear Z	-0.0532
Moment X	-0.1406
Moment Y (Twist)	0.0220
Moment Z	10.7876

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	0.0029	2.6937	0.0000	-0.0000	0.0000	0.0190
ULS: 2. D + L	0.0029	2.6937	0.0000	-0.0000	0.0000	0.0190
ULS: 3. D + (S or Lr or R)	0.0091	7.6898	0.0000	-0.0000	0.0001	0.0074
ULS: 3. D + (S or Lr or R)	0.0029	2.6937	0.0000	-0.0000	0.0000	0.0190
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0076	6.4408	0.0000	-0.0000	0.0001	0.0103
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0029	2.6937	0.0000	-0.0000	0.0000	0.0190
ULS: 5b. D + 0.7E	0.0029	2.6937	0.0000	-0.0000	0.0000	0.0190
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0076	6.4408	0.0000	-0.0000	0.0001	0.0103
ULS: 8. 0.6D + 0.7E	0.0017	1.6162	0.0000	-0.0000	0.0000	0.0114
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5959	6.0926	-0.0000	-0.0000	0.0000	8.0168
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5959	6.0926	-0.0000	-0.0000	0.0000	8.0168
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4235	0.3127	-0.0000	-0.0000	0.0000	-1.5715
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3672	0.6151	-0.0000	-0.0000	0.0000	-10.9510

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	-0.4415	8.9899	0.0000	-0.0000	0.0001	6.0086
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4415	8.9899	0.0000	-0.0000	0.0001	6.0086
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3230	4.6550	0.0000	-0.0000	0.0001	-1.1826
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2808	4.8818	0.0000	-0.0000	0.0001	-8.2172
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4462	5.2429	-0.0000	-0.0000	0.0000	6.0173
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4462	5.2429	-0.0000	-0.0000	0.0000	6.0173
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3184	0.9079	-0.0000	-0.0000	0.0000	-1.1739
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2761	1.1348	-0.0000	-0.0000	0.0000	-8.2085
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5970	5.0151	-0.0000	-0.0000	0.0000	8.0092
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5970	5.0151	-0.0000	-0.0000	0.0000	8.0092
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4224	-0.7648	-0.0000	-0.0000	0.0000	-1.5791
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3660	-0.4624	-0.0000	-0.0000	0.0000	-10.9586

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	14.0588
Shear X	-0.9979
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0004
Moment Z	18.9079

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.9899
Shear X	-0.5970
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0001
Moment Z	10.9586

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	-0.0107	2.7830	0.0148	0.0388	-0.0067	0.1085
ULS: 2. D + L	-0.0107	2.7830	0.0148	0.0388	-0.0067	0.1085
ULS: 3. D + (S or Lr or R)	-0.0338	7.9721	0.0468	0.1230	-0.0212	0.2914
ULS: 3. D + (S or Lr or R)	-0.0107	2.7830	0.0148	0.0388	-0.0067	0.1085
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0281	6.6749	0.0388	0.1020	-0.0176	0.2457
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0107	2.7830	0.0148	0.0388	-0.0067	0.1085
ULS: 5b. D + 0.7E	-0.0107	2.7830	0.0148	0.0388	-0.0067	0.1085
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0281	6.6749	0.0388	0.1020	-0.0176	0.2457
ULS: 8. 0.6D + 0.7E	-0.0064	1.6698	0.0089	0.0233	-0.0040	0.0651
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6234	6.3097	0.0340	0.0903	-0.0089	8.1178
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6234	6.3097	0.0340	0.0903	-0.0089	8.1178
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4246	0.3066	-0.0009	-0.0024	-0.0004	-1.5689
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3549	0.6353	0.0064	0.0152	-0.0124	-10.7442
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4876	9.3198	0.0532	0.1406	-0.0192	6.2527
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4876	9.3198	0.0532	0.1406	-0.0192	6.2527
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2984	4.8175	0.0270	0.0711	-0.0129	-1.0124
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2462	5.0640	0.0325	0.0842	-0.0218	-7.8938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4702	5.4280	0.0292	0.0774	-0.0083	6.1155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4702	5.4280	0.0292	0.0774	-0.0083	6.1155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3158	0.9257	0.0030	0.0079	-0.0020	-1.1496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2635	1.1722	0.0085	0.0211	-0.0110	-8.0310

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6191	5.1964	0.0281	0.0748	-0.0062	8.0744
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6191	5.1964	0.0281	0.0748	-0.0062	8.0744
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4289	-0.8067	-0.0069	-0.0179	0.0023	-1.6123
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3592	-0.4779	0.0005	-0.0004	-0.0097	-10.7876

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	14.5812
Shear X	-1.0448
Shear Z	0.0853
Moment X	0.2257
Moment Y (Twist)	0.0370
Moment Z	18.4969

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	9.3198
Shear X	-0.6234
Shear Z	0.0532
Moment X	0.1406
Moment Y (Twist)	0.0218
Moment Z	10.7876

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0093	2.2673	-0.0922	-0.2327	0.0189	-0.0436
ULS: 2. D + L	0.0093	2.2673	-0.0922	-0.2327	0.0189	-0.0436
ULS: 3. D + (S or Lr or R)	0.0293	6.3441	-0.2917	-0.7368	0.0599	-0.1945
ULS: 3. D + (S or Lr or R)	0.0093	2.2673	-0.0922	-0.2327	0.0189	-0.0436
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0243	5.3249	-0.2418	-0.6107	0.0496	-0.1568
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0093	2.2673	-0.0922	-0.2327	0.0189	-0.0436
ULS: 5b. D + 0.7E	0.0093	2.2673	-0.0922	-0.2327	0.0189	-0.0436
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0243	5.3249	-0.2418	-0.6107	0.0496	-0.1568
ULS: 8. 0.6D + 0.7E	0.0056	1.3604	-0.0553	-0.1396	0.0113	-0.0262
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.4888	5.0384	-0.2354	-0.5897	0.0900	6.5663
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.4888	5.0384	-0.2354	-0.5897	0.0900	6.5663
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3525	0.3259	0.0030	0.0045	-0.0251	-1.3394
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3222	0.5730	0.0016	0.0001	-0.0329	-9.2253
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3493	7.4032	-0.3492	-0.8785	0.1030	4.8007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3493	7.4032	-0.3492	-0.8785	0.1030	4.8007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2817	3.8688	-0.1704	-0.4328	0.0167	-1.1286
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2590	4.0541	-0.1714	-0.4362	0.0108	-7.0430
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3643	4.3457	-0.1996	-0.5005	0.0723	4.9138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3643	4.3457	-0.1996	-0.5005	0.0723	4.9138
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2667	0.8112	-0.0208	-0.0548	-0.0141	-1.0154
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2440	0.9965	-0.0218	-0.0581	-0.0200	-6.9299
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.4925	4.1315	-0.1985	-0.4967	0.0825	6.5837
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.4925	4.1315	-0.1985	-0.4967	0.0825	6.5837
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3488	-0.5811	0.0399	0.0976	-0.0326	-1.3219
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3185	-0.3340	0.0385	0.0932	-0.0405	-9.2079

Worst Case Reactions LRFD

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

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	11.5526
Shear X	-0.8301
Shear Z	-0.5522
Moment X	-1.3949
Moment Y (Twist)	0.1629
Moment Z	15.8892

Result	Value (kip, kip-ft)
Axial	7.4032
Shear X	-0.4925
Shear Z	-0.3492
Moment X	-0.8785
Moment Y (Twist)	0.1030
Moment Z	9.2253

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: MTSOLAR_A262L2JE645HA
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions							

ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
7	6in Pipe Sch 40	6.63	0.28				

Section Dimensions							

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

Section Dimensions							

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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.46,1.03,1.03,1.03,1.11,1.03,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
114	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.05,1.03,1.03,1.04,1.17,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
115	19	8.42	8.42	12.95	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.12,1.17,1.17,1.22,1.14,1.17,1.17,1.18,1.19,1.17,1.17,1.1	300	200	1
116	19	8.42	8.42	12.95	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.25,1.15,1.19,1.19,1.17,1.13,1.18,1.18,1.18,1.17,1.18,1.18,1.2	300	200	1
201	7	18.13	18.13	8.63	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.12,1.17,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
204	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.78,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.7	300	200	1
205	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.67,1.67,1.60,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
206	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.12,1.17,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
207	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.67,1.67,1.60,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
208	19	1.33	1.33	2.05	2.29,2.29,2.29,2.29,2.29,2.29,2.29,2.29,2.07,2.27,2.29,2.29,2.26,2.26,2.29,2.29,2.28,2.29,2.29,2.1	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.78,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.7	300	200	1
211	19	1.33	1.33	2.05	2.28,2.27,2.28,2.27,2.28,2.28,2.28,2.28,2.26,2.25,2.28,2.28,2.19,2.26,2.28,2.28,2.27,2.28,2.28,2.2	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.19,1.03,1.03,1.02,1.06,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
214	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.02,1.03,1.03,1.03,1.03,1.10,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1
215	19	8.42	8.42	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.16,1.17,1.17,1.17,1.17,1.17,1.17,1.1	300	200	1
216	19	8.42	8.42	12.95	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.17,1.16,1.16,1.17,1.18,1.16,1.16,1.16,1.17,1.16,1.16,1.1	300	200	1
301	7	18.13	18.13	8.63	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.12,1.17,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
304	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.77,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.7	300	200	1
305	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.67,1.67,1.61,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
306	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.18,1.18,1.13,1.17,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
307	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.67,1.67,1.61,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
308	19	1.33	1.33	2.05	2.28,2.28,2.28,2.28,2.28,2.28,2.25,2.25,1.78,2.15,2.25,2.25,2.31,2.09,2.27,2.27,2.27,2.30,2.26,2.26,2.1	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.76,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.7	300	200	1
311	19	1.33	1.33	2.05	2.29,2.29,2.29,2.29,2.29,2.29,2.32,2.32,2.32,1.56,2.33,2.33,2.00,2.10,2.30,2.30,2.29,2.24,2.32,2.32,2.3	300	200	1
312	5	1.30	1.30	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.46,1.03,1.03,1.03,1.11,1.03,1.03,1.03,1.03,1.03,1.0	300	200	1

314	19	4.88	4.00	0	2,1.05,1.03,1.03,1.03,1.28	0	0	1
315	19	8.42	8.42	12.95	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.18,1.18,1.22,1.16,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.13,1.18,1.18,1.20,1.16	30	20	1
316	19	8.42	8.42	12.95	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.24,1.17,1.19,1.19,1.18,1.15,1.19,1.19,1.19,1.18,1.19,1.19,1.2,1.17,1.19,1.19,1.18,1.15	30	20	1
401	7	18.13	18.13	8.63	-	30	20	1
402	5	1.30	1.30	2.00	-	30	20	1
403	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.19,1.17,1.18,1.18,1.12,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.2,0,1.16,1.18,1.18,1.17,1.18	30	20	1
404	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.78,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.7,8,1.68,1.67,1.67,1.66,1.68	30	20	1
405	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.67,1.67,1.60,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.6,9,1.64,1.67,1.67,1.65,1.66	30	20	1
406	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.18,1.18,1.06,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1,9,1.13,1.18,1.18,1.15,1.17	30	20	1
407	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.67,1.67,1.57,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.6,9,1.62,1.67,1.67,1.64,1.66	30	20	1
408	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3,3,2.33,2.33,2.33,2.33	30	20	1
409	2	2.60	2.60	4.00	-	30	20	1
410	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.84,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.8,2,1.68,1.67,1.67,1.65,1.68	30	20	1
411	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3,3,2.33,2.33,2.33,2.33	30	20	1
412	5	1.30	1.30	2.00	-	30	20	1
413	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.12,1.41,1.10,1.10,1.06,1.16,1.10,1.10,1.10,1.10,1.10,1.10,1.1,1,1.50,1.10,1.10,1.07,1.13	30	20	1
414	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.13,1.09,1.09,1.12,1.23,1.10,1.10,1.10,1.11,1.10,1.10,1.0,4,1.13,1.09,1.09,1.12,1.32	30	20	1
415	19	8.42	8.42	12.95	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.18,1.12,1.12,1.11,1.13,1.12,1.12,1.12,1.12,1.12,1.1,2,1.19,1.12,1.12,1.12,1.13	30	20	1
416	19	8.42	8.42	12.95	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.14,1.12,1.12,1.12,1.12,1.12,1.1,2,1.12,1.12,1.12,1.12,1.15	30	20	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	251.16	126.44	42.30	42.30	75.35	75.35
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	123.95	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	24.19	6.12	40.24	43.62
14	133.20	85.85	23.92	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	251.16	126.44	42.30	42.30	75.35	75.35
102	198.33	196.72	21.95	21.95	59.50	59.50

103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.44	6.12	40.24	43.62
114	133.20	85.85	23.45	6.12	40.24	43.62
115	133.20	46.28	12.09	6.12	40.24	43.62
116	133.20	46.28	12.16	6.12	40.24	43.62
201	251.16	126.44	42.30	42.30	75.35	75.35
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.47	6.12	40.24	43.62
214	133.20	85.85	23.40	6.12	40.24	43.62
215	133.20	46.28	12.54	6.12	40.24	43.62
216	133.20	46.28	12.45	6.12	40.24	43.62
301	251.16	126.44	42.30	42.30	75.35	75.35
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.44	6.12	40.24	43.62
314	133.20	85.85	23.45	6.12	40.24	43.62
315	133.20	46.28	12.27	6.12	40.24	43.62
316	133.20	46.28	12.41	6.12	40.24	43.62
401	251.16	126.44	42.30	42.30	75.35	75.35
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28

407	116.10	114.23	15.79	11.10	42.08	23.28
408	133.20	52.83	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	52.83	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	24.19	6.12	40.24	43.62
414	133.20	85.85	23.92	6.12	40.24	43.62
415	133.20	46.28	12.07	6.12	40.24	43.62
416	133.20	46.28	12.06	6.12	40.24	43.62

Design Ratio

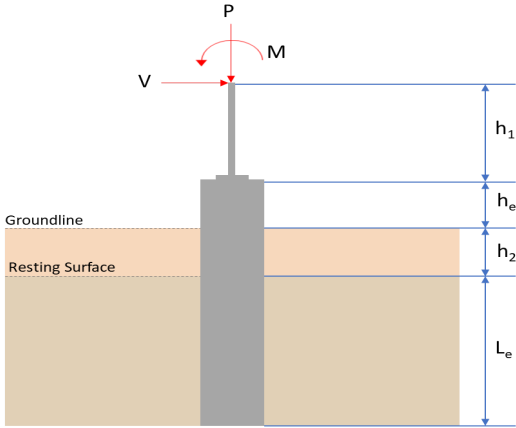
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.091	0.376	0.079	0.011	0.007	0.387	#16	0.484	Not Required	Pass
2	0.003	0.377	0.039	0.084	0.007	0.411	#21	0.035	Not Required	Pass
3	0.004	0.612	0.013	0.061	0.007	0.627	#21	0.045	Not Required	Pass
4	0.002	0.560	0.062	0.056	0.015	0.618	#21	0.080	Not Required	Pass
5	0.003	0.380	0.038	0.061	0.009	0.382	#21	0.074	Not Required	Pass
6	0.004	0.766	0.052	0.078	0.011	0.819	#21	0.045	Not Required	Pass
7	0.004	0.475	0.094	0.076	0.025	0.506	#21	0.074	Not Required	Pass
8	0.001	0.104	0.092	0.050	0.007	0.116	#21	0.095	Not Required	Pass
9	0.006	0.092	0.047	0.003	0.003	0.142	#21	0.204	Not Required	Pass
10	0.005	0.697	0.070	0.070	0.013	0.719	#21	0.080	Not Required	Pass
11	0.003	0.108	0.097	0.055	0.007	0.115	#21	0.095	Not Required	Pass
12	0.001	0.530	0.046	0.107	0.008	0.569	#21	0.053	Not Required	Pass
13	0.005	0.219	0.199	0.068	0.009	0.370	#21	0.286	Not Required	Pass
14	0.004	0.202	0.193	0.062	0.009	0.335	#21	0.190	Not Required	Pass
15	0.000	0.067	0.048	0.029	0.004	0.116	#21	Not Required	Not Required	Pass
16	0.000	0.062	0.048	0.027	0.004	0.110	#21	Not Required	Not Required	Pass
101	0.115	0.437	0.012	0.014	0.001	0.448	#16	0.484	Not Required	Pass
102	0.002	0.593	0.057	0.123	0.009	0.645	#21	0.035	Not Required	Pass
103	0.004	0.881	0.024	0.089	0.001	0.908	#21	0.045	Not Required	Pass
104	0.004	0.818	0.081	0.082	0.016	0.867	#21	0.080	Not Required	Pass
105	0.004	0.547	0.090	0.088	0.023	0.573	#21	0.074	Not Required	Pass
106	0.004	0.861	0.019	0.086	0.002	0.879	#21	0.045	Not Required	Pass
107	0.004	0.535	0.083	0.086	0.021	0.557	#21	0.074	Not Required	Pass
108	0.001	0.091	0.090	0.055	0.007	0.181	#21	0.095	Not Required	Pass
109	0.009	0.097	0.025	0.001	0.000	0.126	#21	0.204	Not Required	Pass
110	0.004	0.793	0.084	0.080	0.018	0.850	#21	0.080	Not Required	Pass
111	0.003	0.093	0.091	0.059	0.007	0.185	#21	0.095	Not Required	Pass
112	0.002	0.570	0.056	0.119	0.009	0.620	#21	0.035	Not Required	Pass
113	0.005	0.347	0.199	0.076	0.009	0.525	#21	0.286	Not Required	Pass
114	0.006	0.338	0.199	0.071	0.009	0.511	#21	0.286	Not Required	Pass
115	0.009	0.582	0.103	0.063	0.007	0.690	#21	0.601	Not Required	Pass
116	0.004	0.532	0.106	0.059	0.007	0.639	#21	0.601	Not Required	Pass
201	0.111	0.447	0.000	0.013	0.000	0.456	#16	0.484	Not Required	Pass
202	0.002	0.560	0.053	0.116	0.008	0.606	#21	0.035	Not Required	Pass
203	0.004	0.844	0.021	0.085	0.000	0.867	#21	0.045	Not Required	Pass
204	0.004	0.770	0.079	0.078	0.016	0.822	#21	0.080	Not Required	Pass
205	0.004	0.524	0.084	0.084	0.021	0.547	#21	0.074	Not Required	Pass

203	0.004	0.524	0.004	0.004	0.021	0.547	#21	0.074	Not Required	Pass
206	0.004	0.844	0.021	0.085	0.000	0.867	#21	0.045	Not Required	Pass
207	0.004	0.524	0.084	0.084	0.021	0.547	#21	0.074	Not Required	Pass
208	0.001	0.073	0.087	0.053	0.007	0.160	#21	0.095	Not Required	Pass
209	0.008	0.089	0.021	0.001	0.000	0.114	#21	0.204	Not Required	Pass
210	0.004	0.770	0.079	0.078	0.016	0.822	#21	0.080	Not Required	Pass
211	0.003	0.083	0.089	0.058	0.007	0.173	#21	0.095	Not Required	Pass
212	0.002	0.560	0.053	0.116	0.008	0.606	#21	0.035	Not Required	Pass
213	0.004	0.329	0.189	0.072	0.009	0.501	#21	0.286	Not Required	Pass
214	0.006	0.306	0.187	0.066	0.009	0.467	#21	0.286	Not Required	Pass
215	0.008	0.467	0.104	0.058	0.007	0.575	#21	0.601	Not Required	Pass
216	0.004	0.427	0.104	0.053	0.007	0.532	#21	0.601	Not Required	Pass
301	0.115	0.437	0.012	0.014	0.001	0.448	#16	0.484	Not Required	Pass
302	0.002	0.570	0.056	0.119	0.009	0.620	#21	0.035	Not Required	Pass
303	0.004	0.861	0.019	0.086	0.002	0.879	#21	0.045	Not Required	Pass
304	0.004	0.793	0.084	0.080	0.018	0.850	#21	0.080	Not Required	Pass
305	0.004	0.535	0.083	0.086	0.021	0.557	#21	0.074	Not Required	Pass
306	0.004	0.881	0.024	0.089	0.001	0.908	#21	0.045	Not Required	Pass
307	0.004	0.547	0.090	0.088	0.023	0.573	#21	0.074	Not Required	Pass
308	0.001	0.084	0.097	0.059	0.007	0.181	#21	0.095	Not Required	Pass
309	0.009	0.097	0.025	0.001	0.000	0.126	#21	0.204	Not Required	Pass
310	0.004	0.818	0.081	0.082	0.016	0.867	#21	0.080	Not Required	Pass
311	0.003	0.083	0.097	0.063	0.007	0.182	#21	0.095	Not Required	Pass
312	0.002	0.593	0.057	0.123	0.009	0.645	#21	0.035	Not Required	Pass
313	0.005	0.347	0.199	0.076	0.009	0.525	#21	0.286	Not Required	Pass
314	0.006	0.338	0.199	0.071	0.009	0.511	#21	0.286	Not Required	Pass
315	0.008	0.464	0.104	0.059	0.007	0.572	#21	0.601	Not Required	Pass
316	0.004	0.422	0.104	0.055	0.007	0.527	#21	0.601	Not Required	Pass
401	0.091	0.376	0.079	0.011	0.007	0.387	#16	0.484	Not Required	Pass
402	0.001	0.530	0.046	0.107	0.008	0.569	#21	0.053	Not Required	Pass
403	0.004	0.766	0.052	0.078	0.011	0.819	#21	0.045	Not Required	Pass
404	0.005	0.697	0.070	0.070	0.013	0.719	#21	0.080	Not Required	Pass
405	0.004	0.475	0.094	0.076	0.025	0.506	#21	0.074	Not Required	Pass
406	0.004	0.612	0.013	0.061	0.007	0.627	#21	0.045	Not Required	Pass
407	0.003	0.380	0.038	0.061	0.009	0.382	#21	0.074	Not Required	Pass
408	0.000	0.062	0.048	0.027	0.004	0.110	#21	Not Required	Not Required	Pass
409	0.006	0.092	0.047	0.003	0.003	0.142	#21	0.204	Not Required	Pass
410	0.002	0.560	0.062	0.056	0.015	0.618	#21	0.120	Not Required	Pass
411	0.000	0.067	0.048	0.029	0.004	0.116	#21	Not Required	Not Required	Pass
412	0.003	0.377	0.039	0.084	0.007	0.411	#21	0.035	Not Required	Pass
413	0.005	0.219	0.199	0.068	0.009	0.370	#21	0.190	Not Required	Pass
414	0.004	0.202	0.193	0.062	0.009	0.335	#21	0.286	Not Required	Pass
415	0.009	0.606	0.104	0.055	0.007	0.711	#21	0.601	Not Required	Pass
416	0.004	0.559	0.105	0.050	0.007	0.664	#21	0.601	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																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: round $D = 36$ in - Pile diameter $L = 7$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1061 1227 1162"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.403</td> <td>11.553</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.493</td> <td>-0.830</td> </tr> <tr> <td>V_z (kip)</td> <td>0.349</td> <td>0.552</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.878</td> <td>1.395</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.225</td> <td>15.889</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	7.403	11.553	V_x (kip)	-0.493	-0.830	V_z (kip)	0.349	0.552	M_x (kipft)	0.878	1.395	M_z (kipft)	9.225	15.889	
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M_x (kipft)	0.878	1.395																										
M_z (kipft)	9.225	15.889																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.493 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.16433 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(9.225 \text{ kipft}) + ((-0.493 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.075 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.5773 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.349 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.11633 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.878 \text{ kipft}) + ((0.349 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.29267 \text{ kipft/ft}$$

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:

$L_{e,z} = 4.3967 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.5773 \text{ ft}), (4.3967 \text{ ft})]$$

$$L_{e,req} = 6.577 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.577 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.93957$$

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

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

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

$$q = \frac{(7.403 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0473 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.0473 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.52366$$

Status: **PASS**
Ratio: **0.520**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16433 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.075 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.075 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (3.075 \text{ kipft/ft})) + (4 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7831 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.075 \text{ kipft/ft})) + (3 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (3.075 \text{ kipft/ft})) + (2 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.27187 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.075 \text{ kipft/ft})) + ((-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.96167 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7831 \text{ ft})}{2}$$

$$p_a = 0.35873 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27187 \text{ kip/ft}^2)}{(0.35873 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75786$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.96167 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91588$$

Status: **PASS**
Ratio: **0.760**

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

Considering z-direction:

$H_o = 0.11633 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.29267 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.29267 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.11633 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.29267 \text{ kipft/ft})) + (4 \times (0.11633 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0457 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.29267 \text{ kipft/ft})) + (3 \times (0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.29267 \text{ kipft/ft})) + (2 \times (0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.12524 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.29267 \text{ kipft/ft})) + ((0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.26922 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0457 \text{ ft})}{2}$$

$$p_a = 0.37843 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.12524 \text{ kip/ft}^2)}{(0.37843 \text{ kip/ft}^2)}$$

$$(0.0126 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.33095$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

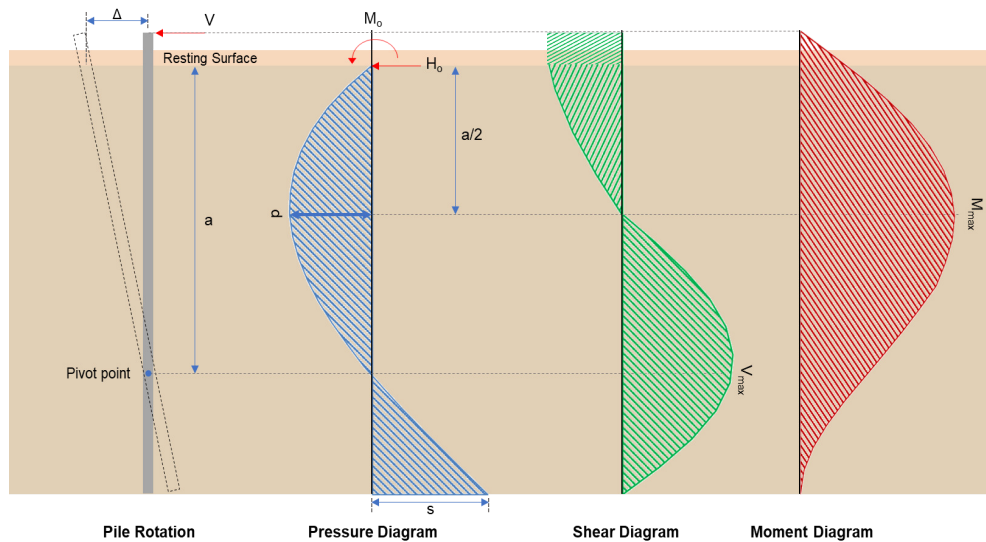
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.26922 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.2564$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.83 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.27667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.889 \text{ kipft}) + ((-0.83 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.2963 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.2963 \text{ kipft/ft})}{(-0.27667 \text{ kip/ft})}$$

$$E = 19.143 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.2963 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.27667 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.2963 \text{ kipft/ft})) + (4 \times (-0.27667 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.781 \text{ ft}$$

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

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

$$V_{max} = ((-0.27667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.781 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.781 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.567 \text{ kip}$$

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

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

$$M_{max} = ((-0.27667 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(19.143 \text{ ft})}{(7 \text{ ft})} + \frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 15.454 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.552 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.184 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.395 \text{ kipft}) + ((0.552 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.465 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.465 \text{ kipft/ft})}{(0.184 \text{ kip/ft})}$$

$$E = 2.5272 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.465 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.184 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.465 \text{ kipft/ft})) + (4 \times (0.184 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0451 \text{ ft}$$

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

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

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

$$V_{max} = ((0.184 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0451 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0451 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.72227 \text{ kip}$$

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

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

$$M_{max} = ((0.184 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(2.5272 \text{ ft})}{(7 \text{ ft})} + \frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.1847 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.553 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.012 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.012 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

<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;">$Ratio = \frac{(11.553 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0092136$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.553 \text{ kip} \rightarrow 11553 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(11553 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.399 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.399 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.399 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.399 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.47 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.567 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.567 \text{ kip})}{(74.47 \text{ kip})}$$

$$Ratio = 0.061327$$

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

Considering z-direction:

$V_{max} = 0.72227 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.72227 \text{ kip})}{(74.47 \text{ kip})}$$

$$Ratio = 0.0096988$$

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:

$M_{max} = 15.454 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.454 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.24915$$

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

Considering z-direction:

$M_{max} = 2.1847 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

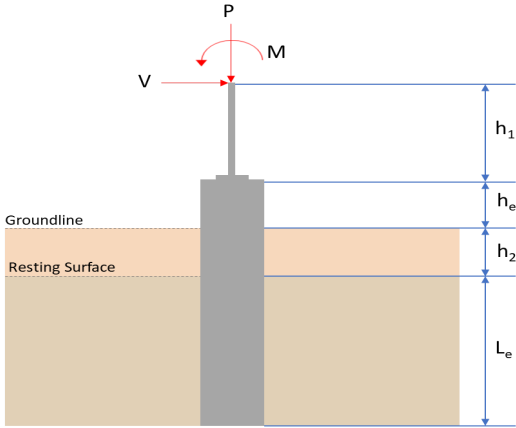
$$Ratio = \frac{M_{max}}{\phi M_n}$$

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

$$Ratio = \frac{(2.1847 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.035223$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: round $D = 36$ in - Pile diameter $L = 7$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1061 1227 1162"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.403</td> <td>11.553</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.493</td> <td>-0.830</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.349</td> <td>-0.552</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.879</td> <td>-1.395</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.225</td> <td>15.889</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	7.403	11.553	V_x (kip)	-0.493	-0.830	V_z (kip)	-0.349	-0.552	M_x (kipft)	-0.879	-1.395	M_z (kipft)	9.225	15.889	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.493 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.16433 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(9.225 \text{ kipft}) + ((-0.493 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.075 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.5773 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.349 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.11633 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.879 \text{ kipft}) + ((-0.349 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.293 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2787 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.5773 \text{ ft}), (2.2787 \text{ ft})]$$

$$L_{e,req} = 6.577 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.577 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.93957$$

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

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

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

$$q = \frac{(7.403 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0473 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.0473 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.52366$$

Status: **PASS**
Ratio: **0.520**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16433 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.075 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.075 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (3.075 \text{ kipft/ft})) + (4 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7831 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.075 \text{ kipft/ft})) + (3 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (3.075 \text{ kipft/ft})) + (2 \times (-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.27187 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.075 \text{ kipft/ft})) + ((-0.16433 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.96167 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7831 \text{ ft})}{2}$$

$$p_a = 0.35873 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27187 \text{ kip/ft}^2)}{(0.35873 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75786$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.96167 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91588$$

Status: **PASS**
Ratio: **0.760**

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

Considering z-direction:

$H_o = -0.11633 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.293 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.11633 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.293 \text{ kipft/ft})) + (4 \times (-0.11633 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0455 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.293 \text{ kipft/ft})) + (3 \times (-0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.293 \text{ kipft/ft})) + (2 \times (-0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = -0.051805 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.293 \text{ kipft/ft})) + ((-0.11633 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = -0.043919 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0455 \text{ ft})}{2}$$

$$p_a = 0.37841 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.051805 \text{ kip/ft}^2)}{(0.37841 \text{ kip/ft}^2)}$$

(0.043919 kip/ft²)

$$\text{Ratio} = -0.1369$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

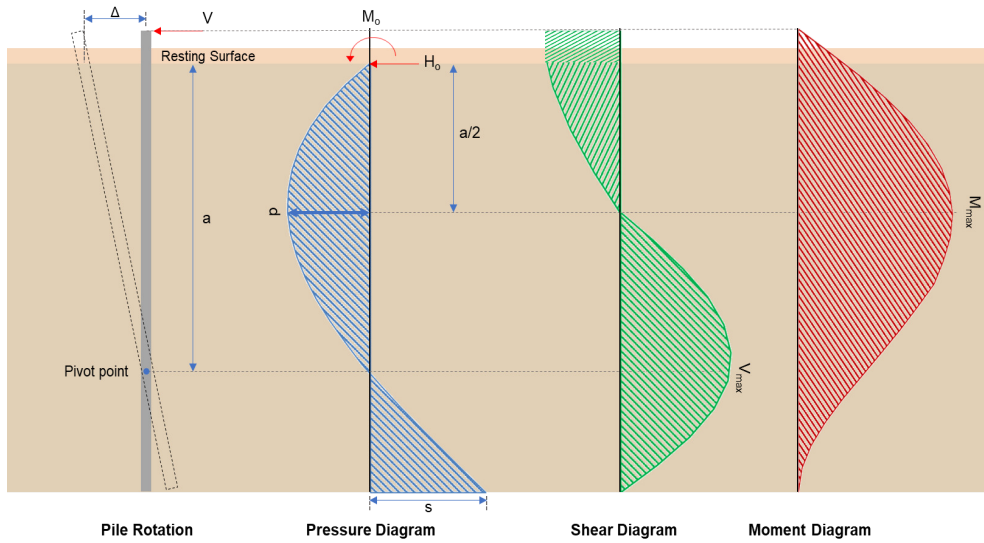
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.043919 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.041828$$

Status: **PASS**
Ratio: **-0.140**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.83 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.27667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.889 \text{ kipft}) + ((-0.83 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.2963 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.2963 \text{ kipft/ft})}{(-0.27667 \text{ kip/ft})}$$

$$E = 19.143 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.2963 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.27667 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.2963 \text{ kipft/ft})) + (4 \times (-0.27667 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.781 \text{ ft}$$

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

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

$$V_{max} = ((-0.27667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.781 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.781 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.567 \text{ kip}$$

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

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

$$M_{max} = ((-0.27667 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(19.143 \text{ ft})}{(7 \text{ ft})} + \frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.143 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.781 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 15.454 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.552 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.184 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.395 \text{ kipft}) + ((-0.552 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.465 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.465 \text{ kipft/ft})}{(-0.184 \text{ kip/ft})}$$

$$E = 2.5272 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.465 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.184 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.465 \text{ kipft/ft})) + (4 \times (-0.184 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0451 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.184 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0451 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0451 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.72227 \text{ kip}$$

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

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

$$M_{max} = ((-0.184 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(2.5272 \text{ ft})}{(7 \text{ ft})} + \frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5272 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0451 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.1847 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.553 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.012 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.012 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

<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>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.553 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0092136$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.553 \text{ kip} \rightarrow 11553 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(11553 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.399 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.399 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.399 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.399 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.47 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.567 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.567 \text{ kip})}{(74.47 \text{ kip})}$$

$$Ratio = 0.061327$$

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

Considering z-direction:

$V_{max} = 0.72227 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.72227 \text{ kip})}{(74.47 \text{ kip})}$$

$$Ratio = 0.0096988$$

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:

$M_{max} = 15.454 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.454 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.24915$$

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

Considering z-direction:

$M_{max} = 2.1847 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(2.1847 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.035223$$

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

REFERENCES	CALCULATIONS	RESULTS
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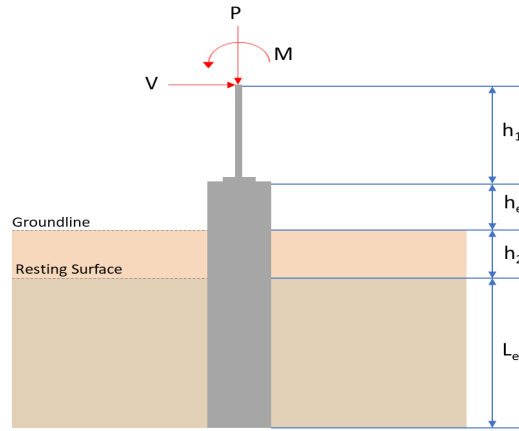
SkyCiv Foundation Design

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

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.320	14.581
V_x (kip)	-0.623	-1.045
V_z (kip)	-0.053	-0.085
M_x (kipft)	-0.141	-0.226
M_z (kipft)	10.788	18.497

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,

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

$$H_o = \frac{(-0.623 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.20767 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.788 \text{ kipft}) + ((-0.623 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.596 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8274 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.053 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.017667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.141 \text{ kipft}) + ((-0.053 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.047 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5039 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.8274 \text{ ft}), (1.5039 \text{ ft})]$$

$$L_{e,req} = 6.827 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.827 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.94166$$

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

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

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

$$q = \frac{(9.32 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.3185 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.3185 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.65926$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.4167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.20767 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.596 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (3.596 \text{ kipft/ft})) + (4 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9652 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.596 \text{ kipft/ft})) + (3 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (3.596 \text{ kipft/ft})) + (2 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.28058 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.596 \text{ kipft/ft})) + ((-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0196 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9652 \text{ ft})}{2}$$

$$p_a = 0.37239 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28058 \text{ kip/ft}^2)}{(0.37239 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75347$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0196 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93759$$

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

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

Considering z-direction:

$H_o = -0.017667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.047 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.047 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.017667 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.047 \text{ kipft/ft})) + (4 \times (-0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.223 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.047 \text{ kipft/ft})) + (3 \times (-0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.047 \text{ kipft/ft})) + (2 \times (-0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.0074948 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.047 \text{ kipft/ft})) + ((-0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.0061115 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.223 \text{ ft})}{2}$$

$$p_a = 0.39173 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0074948 \text{ kip/ft}^2)}{(0.39173 \text{ kip/ft}^2)}$$

$$(-0.00115 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.019133$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

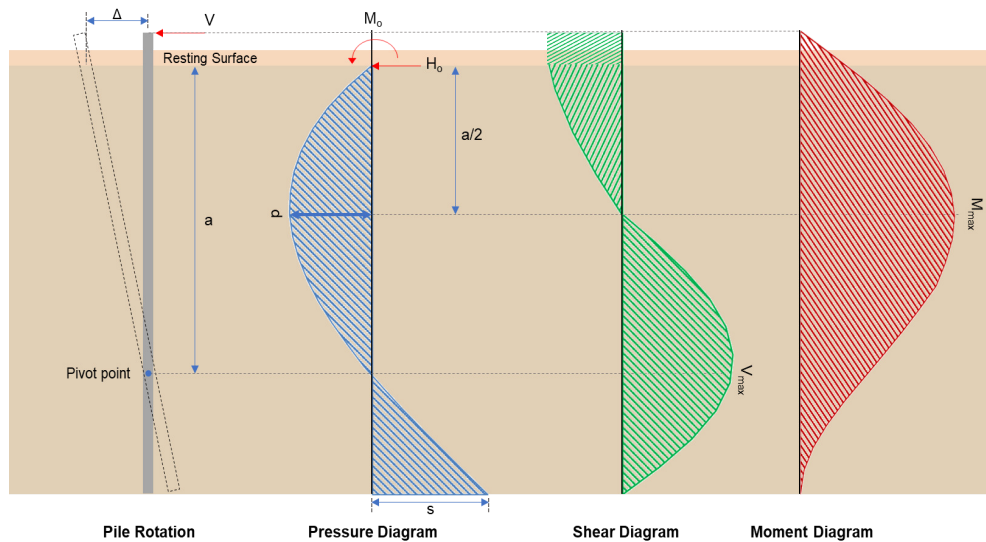
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0061115 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0056198$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.045 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.34833 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.497 \text{ kipft}) + ((-1.045 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.1657 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.1657 \text{ kipft/ft})}{(-0.34833 \text{ kip/ft})}$$

$$E = 17.7 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1657 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.34833 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.1657 \text{ kipft/ft})) + (4 \times (-0.34833 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9629 \text{ ft}$$

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

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

$$V_{max} = ((-0.34833 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9629 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9629 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.2062 \text{ kip}$$

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

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

$$M_{max} = ((-0.34833 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(17.7 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.182 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.085 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.028333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.226 \text{ kipft}) + ((-0.085 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.075333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.075333 \text{ kipft/ft})}{(-0.028333 \text{ kip/ft})}$$

$$E = 2.6588 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075333 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.028333 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.075333 \text{ kipft/ft})) + (4 \times (-0.028333 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2231 \text{ ft}$$

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

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

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

$$V_{max} = ((-0.028333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.2231 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.2231 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.11206 \text{ kip}$$

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

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

$$M_{max} = ((-0.028333 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(2.6588 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.2231 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.2231 \text{ ft})}{(2 \times (7.25 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.2231 \text{ ft})}{(2 \times (7.25 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 0.35149 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.581 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.917 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.917 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

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;">$Ratio = \frac{(14.581 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.011628$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.581 \text{ kip} \rightarrow 14581 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(14581 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.913 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.913 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.913 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((76.913 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.804 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.2062 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.2062 \text{ kip})}{(74.804 \text{ kip})}$$

$$Ratio = 0.069597$$

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

Considering z-direction:

$V_{max} = 0.11206 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.11206 \text{ kip})}{(74.804 \text{ kip})}$$

$$Ratio = 0.0014981$$

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:

$M_{max} = 18.182 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.182 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.29313$$

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

Considering z-direction:

$M_{max} = 0.35149 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.35149 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0056667$$

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

REFERENCES	CALCULATIONS	RESULTS
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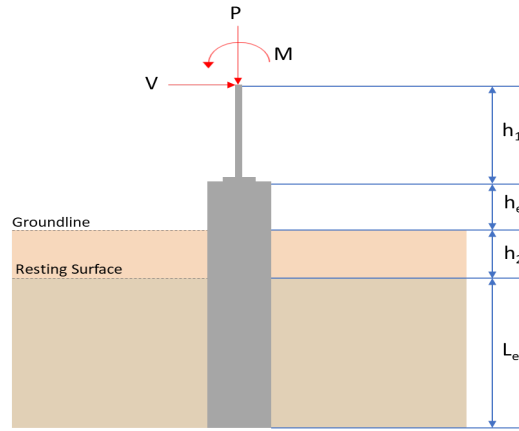
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.990	14.059
V_x (kip)	-0.597	-0.998
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	10.959	18.908

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,

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

$$H_o = \frac{(-0.597 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.199 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.959 \text{ kipft}) + ((-0.597 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.653 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.9069 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9069 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.907 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.907 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.95269$$

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

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

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

$$q = \frac{(8.99 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2718 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.2718 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.63591$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.4167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.199 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.653 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.653 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.199 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (3.653 \text{ kipft/ft})) + (4 \times (-0.199 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9593 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.653 \text{ kipft/ft})) + (3 \times (-0.199 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (3.653 \text{ kipft/ft})) + (2 \times (-0.199 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.29357 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.653 \text{ kipft/ft})) + ((-0.199 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0513 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9593 \text{ ft})}{2}$$

$$p_a = 0.37194 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29357 \text{ kip/ft}^2)}{(0.37194 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.78928$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

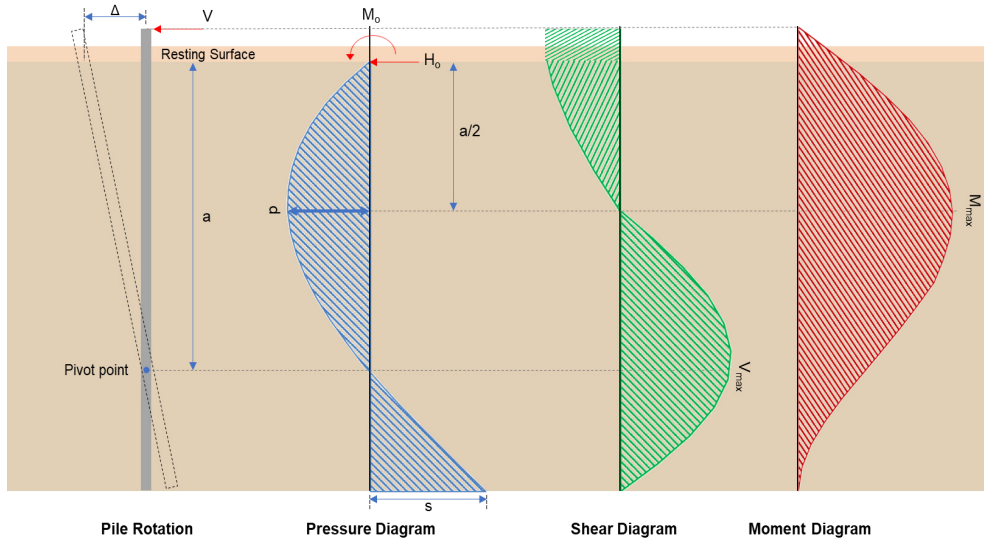
$$(1.0513 \text{ kip/ft}^2)$$

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

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

$$\text{Ratio} = 0.96675$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.998 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.33267 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.908 \text{ kipft}) + ((-0.998 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.3027 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3027 \text{ kipft/ft})}{(-0.33267 \text{ kip/ft})}$$

$$E = 18.946 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3027 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.33267 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.3027 \text{ kipft/ft})) + (4 \times (-0.33267 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9561 \text{ ft}$$

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

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

$$V_{max} = ((-0.33267 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (18.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9561 \text{ ft})}{(7.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (18.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9561 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.2762 \text{ kip}$$

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

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

$$M_{max} = ((-0.33267 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(18.946 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9561 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (18.946 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9561 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (18.946 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9561 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.466 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.059 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.934 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.934 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

Status: **PASS**
Ratio: **1.000**

<p>25.7.2.2 25.7.2.1</p>	$s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties: Since longitudinal reinforcement is \leq No. 10e: Use #3(0.375 in) s_{ties} - Maximum center-to-center spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p>Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$ $\phi P_N = 1253.9 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(14.059 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.011212$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters: $b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.059 \text{ kip} \rightarrow 14059 \text{ lbf}$,</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(14059 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.825 \text{ kip}$$

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

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.825 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.825 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((76.825 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.747 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.2762 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(5.2762 \text{ kip})}{(74.747 \text{ kip})}$$

$$\text{Ratio} = 0.070587$$

Status: **PASS**
 Ratio: **0.070587**

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 $\phi M_{n,2}$

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

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

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

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: $M_{max} = 18.466 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(18.466 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.29771$$

REFERENCES	CALCULATIONS	RESULTS
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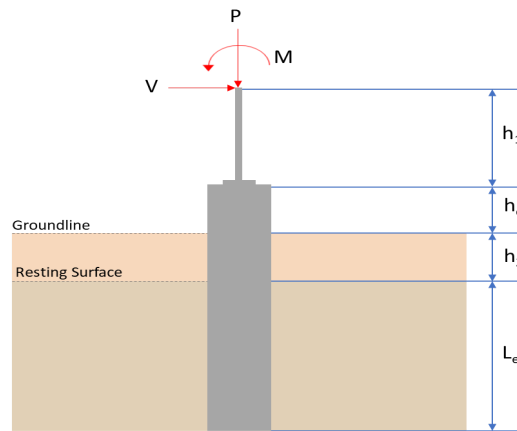
SkyCiv Foundation Design

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

$L = 7.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.320	14.581
V_x (kip)	-0.623	-1.045
V_z (kip)	0.053	0.085
M_x (kipft)	0.141	0.226
M_z (kipft)	10.788	18.497

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,

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

$$H_o = \frac{(-0.623 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.20767 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.788 \text{ kipft}) + ((-0.623 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.596 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8274 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.053 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.017667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.141 \text{ kipft}) + ((0.053 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.047 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1123 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.8274 \text{ ft}), (2.1123 \text{ ft})]$$

$$L_{e,req} = 6.827 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.827 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.94166$$

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

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

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

$$q = \frac{(9.32 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.3185 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.3185 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.65926$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.4167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.20767 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.596 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (3.596 \text{ kipft/ft})) + (4 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9652 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.596 \text{ kipft/ft})) + (3 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (3.596 \text{ kipft/ft})) + (2 \times (-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.28058 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.596 \text{ kipft/ft})) + ((-0.20767 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0196 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9652 \text{ ft})}{2}$$

$$p_a = 0.37239 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28058 \text{ kip/ft}^2)}{(0.37239 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75347$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0196 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93759$$

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

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

Considering z-direction:

$H_o = 0.017667 \text{ kip/ft}$ - Lateral force per length of pile.

$M_o = 0.047 \text{ kipft/ft}$ - Overturning moment per length of pile.

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.047 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.017667 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.047 \text{ kipft/ft})) + (4 \times (0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.223 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.047 \text{ kipft/ft})) + (3 \times (0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.047 \text{ kipft/ft})) + (2 \times (0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.018479 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.047 \text{ kipft/ft})) + ((0.017667 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.039822 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.223 \text{ ft})}{2}$$

$$p_a = 0.39173 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.018479 \text{ kip/ft}^2)}{(0.39173 \text{ kip/ft}^2)}$$

$$(0.047172 \text{ kip/ft})$$

$$\text{Ratio} = 0.047172$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

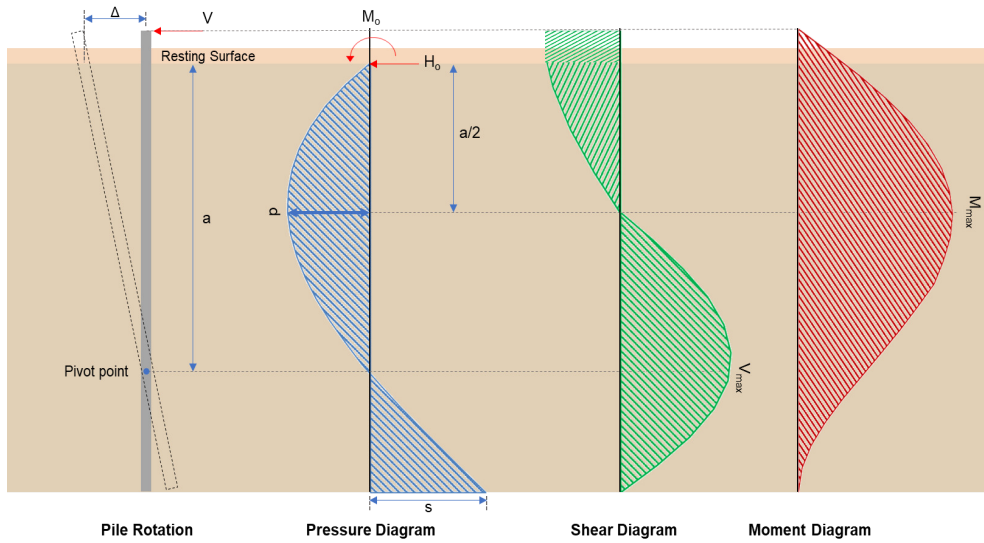
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.039822 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.036618$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.045 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.34833 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.497 \text{ kipft}) + ((-1.045 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.1657 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.1657 \text{ kipft/ft})}{(-0.34833 \text{ kip/ft})}$$

$$E = 17.7 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.1657 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.34833 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (6.1657 \text{ kipft/ft})) + (4 \times (-0.34833 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9629 \text{ ft}$$

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

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

$$V_{max} = ((-0.34833 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9629 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9629 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.2062 \text{ kip}$$

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

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

$$M_{max} = ((-0.34833 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(17.7 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.7 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9629 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.182 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.085 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.028333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.226 \text{ kipft}) + ((0.085 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.075333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.075333 \text{ kipft/ft})}{(0.028333 \text{ kip/ft})}$$

$$E = 2.6588 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075333 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.028333 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.075333 \text{ kipft/ft})) + (4 \times (0.028333 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2231 \text{ ft}$$

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

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

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

$$V_{max} = ((0.028333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.2231 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.2231 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.11206 \text{ kip}$$

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

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

$$M_{max} = ((0.028333 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(2.6588 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.2231 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.2231 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6588 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.2231 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.35149 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.581 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.917 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.917 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

<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;">$Ratio = \frac{(14.581 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.011628$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.581 \text{ kip} \rightarrow 14581 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(14581 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.913 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.913 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.913 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((76.913 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.804 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.2062 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.2062 \text{ kip})}{(74.804 \text{ kip})}$$

$$Ratio = 0.069597$$

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

Considering z-direction:

$V_{max} = 0.11206 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.11206 \text{ kip})}{(74.804 \text{ kip})}$$

$$Ratio = 0.0014981$$

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

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:

$M_{max} = 18.182 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.182 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.29313$$

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

Considering z-direction:

$M_{max} = 0.35149 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

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

$$Ratio = \frac{(0.35149 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0056667$$

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