

Project Name: Norris Pool House - 4x11 - 60INsquare - **Date:** Mon Dec 30 2024

V1Jb

Number of Modules: 44

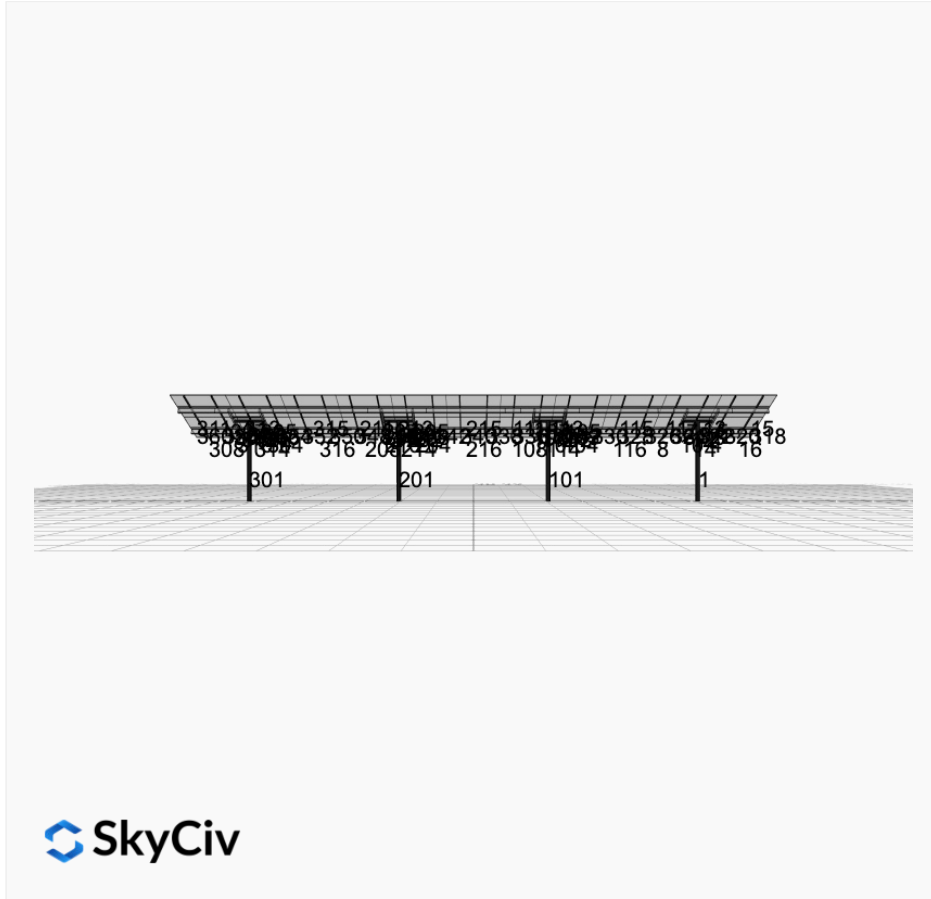
Location: 52 MT-84, Norris, MT 59745, USA

Number of Poles: 4

Unique ID: 4P-19.75-6TOP-HD-57-L-4Hx11W-0CB8

Date Sold:

Dealer: _____



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	76.63 ft
Winter Tilt Angle	20
Front Edge Clearance	8 ft

MT Solar Bill of Materials (4P-19.75-6TOP-HD-57-L-4Hx11W-0CB8)

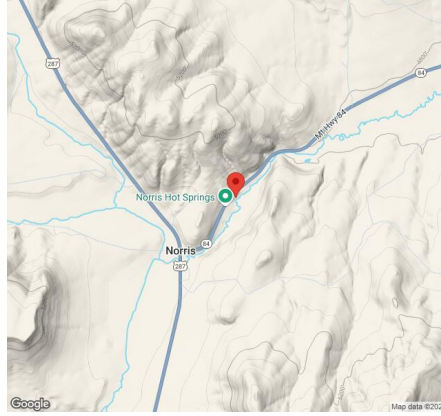
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-57	57IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	6
MTS-HD-Splice-57	57IN HD Splice	6
MTS-CLAMP-HOOK-4PK	Hook Clamp	11

Rail Bill of Materials

Part	Qty
Rails (178in)	22
Rail Attachment	44

Part	Qty
Module Mid Clamp	66
Module End Clamp	44
Ground Lug	11

Site Details:



Site Address: 52 MT-84, Norris, MT 59745, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	4
Number of Columns:	11
Total Number of Modules:	44
Winter Tilt Angle:	20
Front Edge Clearance:	8
Total Array Height at Tilt:	13.14 ft
Total Frame Length:	76.25 ft
Frame Weight:	4118 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	76.63 ft
Rail Length:	180.40 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.57 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 5.50 ft Pile 2: 5.75 ft Pile 3: 5.75 ft Pile 4: 5.50 ft
Foundation Volume:	20.833 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	52 MT-84, Norris, MT 59745, USA
Wind Speed:	106 mph

Snow Load:

36 psf

Design Disclaimer

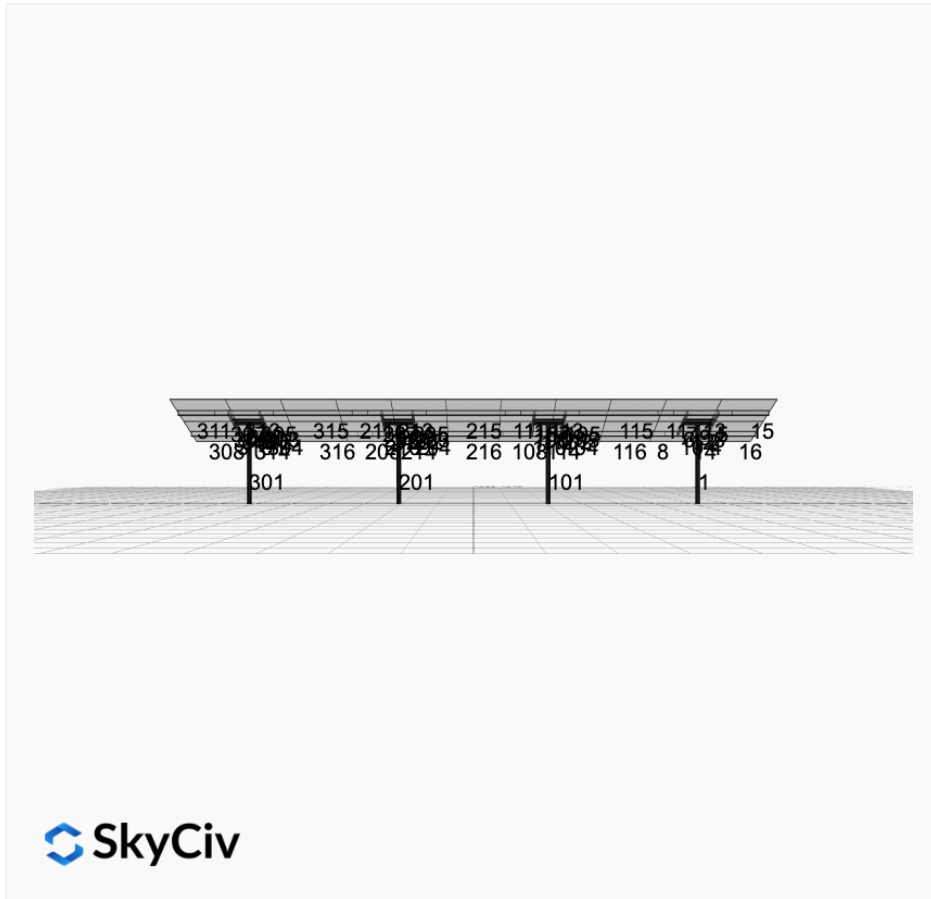
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

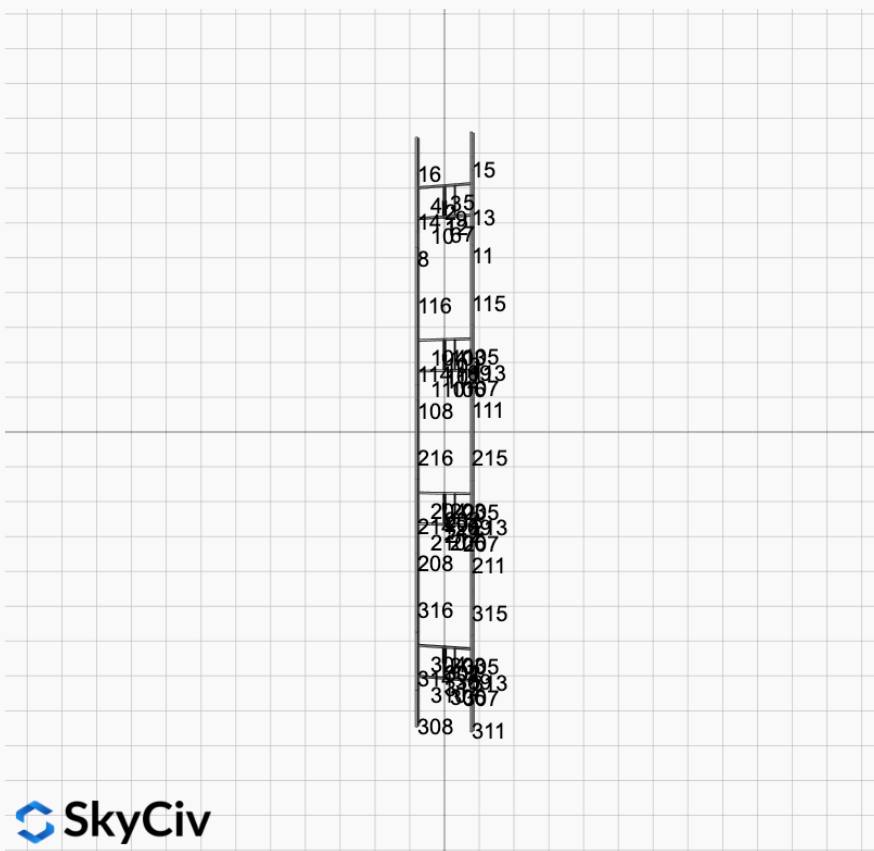
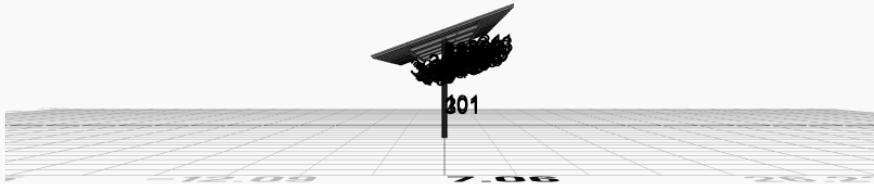
AutoDesigner Input

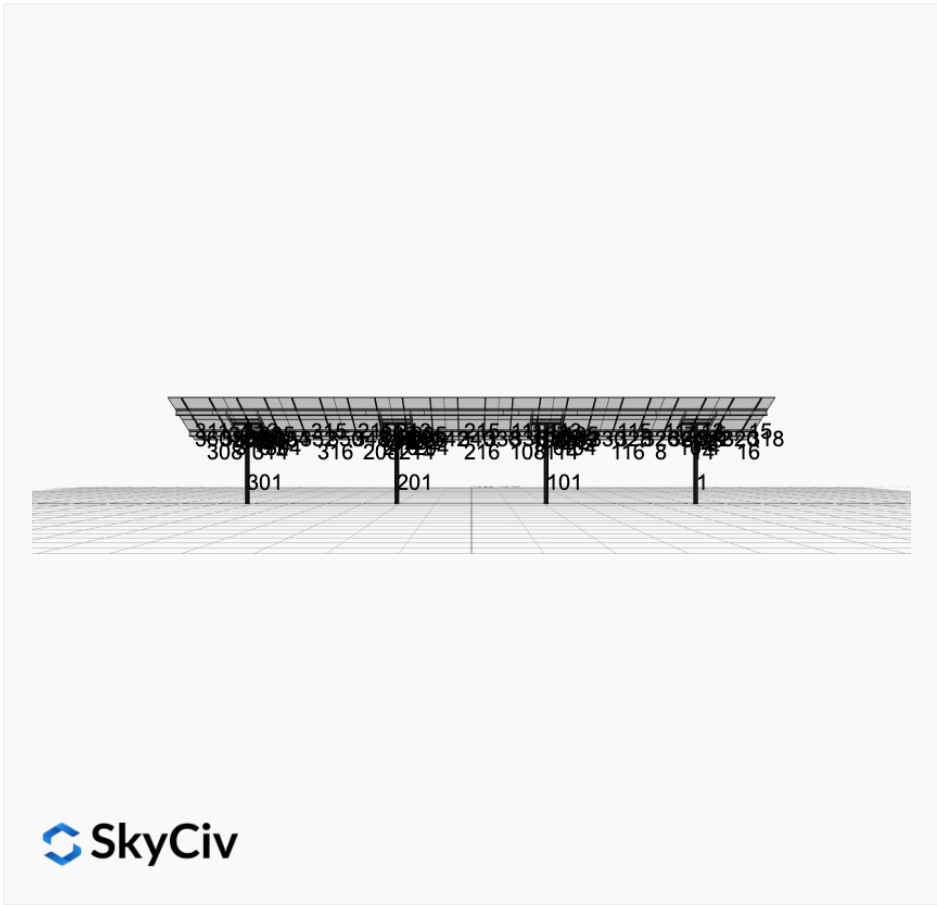
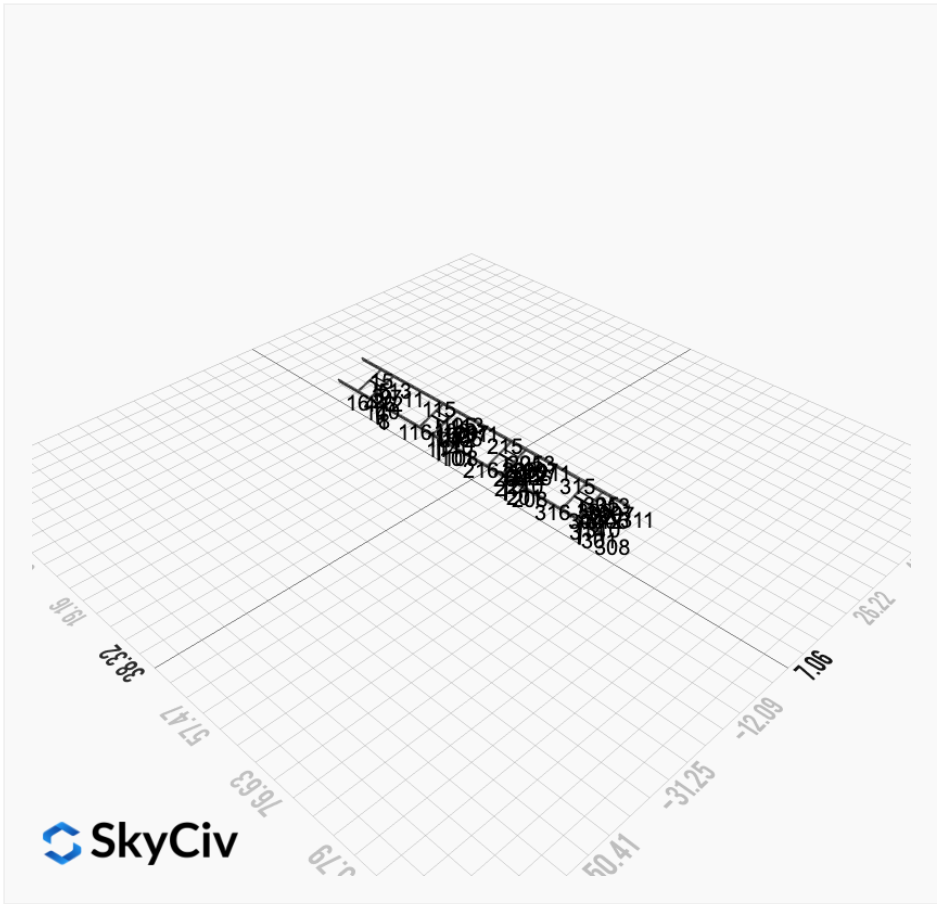
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Design Notes:

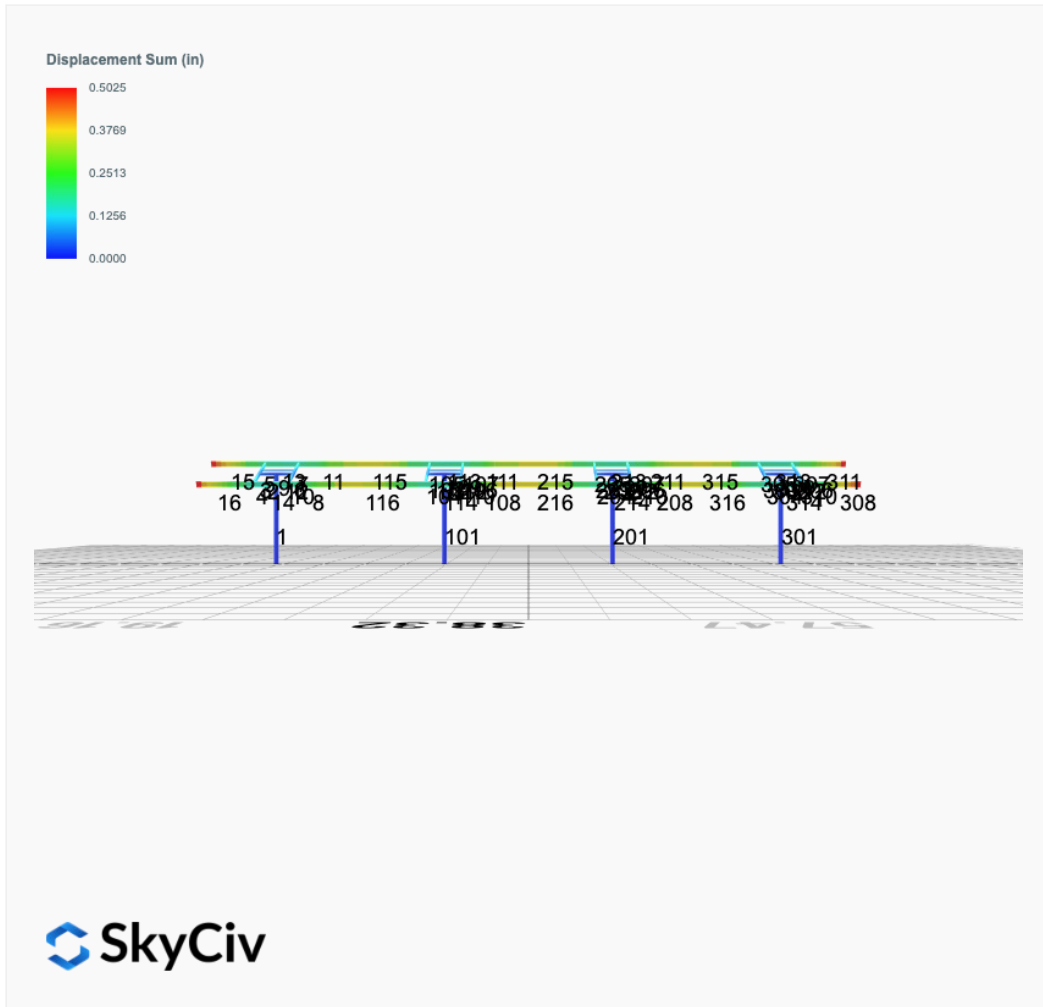
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



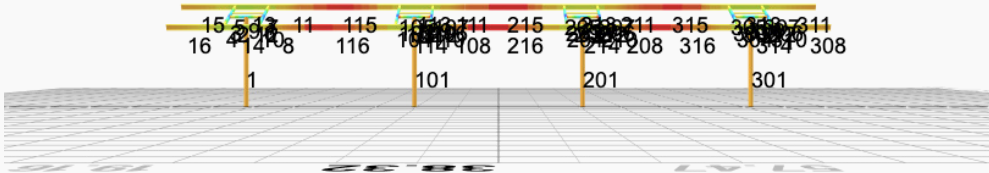




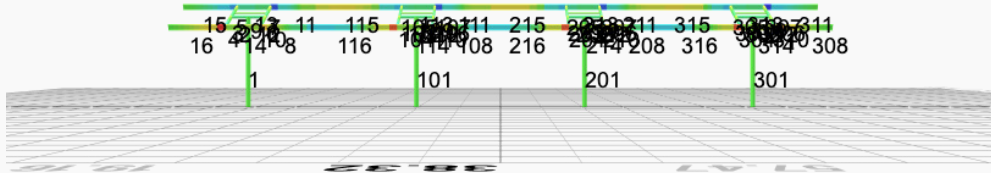
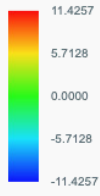
FEM Results (Envelope Worst Case for each member)



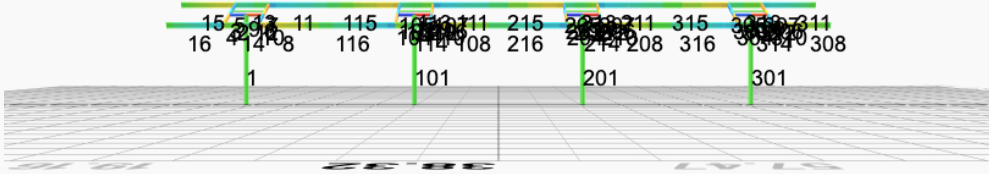
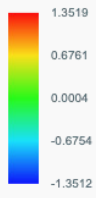
Top Bending Stress Z (ksi)



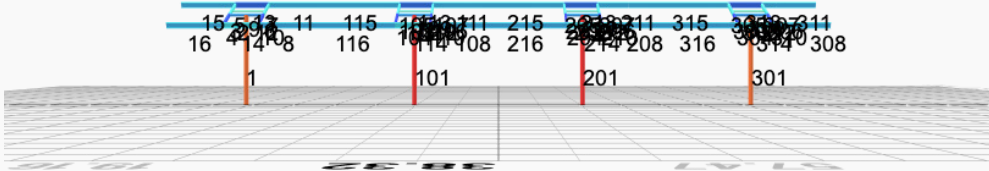
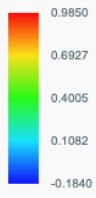
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0024	2.0606	-0.0061	-0.0197	0.0102	0.0504
ULS: 2. D + L	-0.0024	2.0606	-0.0061	-0.0197	0.0102	0.0504
ULS: 3. D + (S or Lr or R)	-0.0096	7.2242	-0.0247	-0.0801	0.0418	0.1427
ULS: 3. D + (S or Lr or R)	-0.0024	2.0606	-0.0061	-0.0197	0.0102	0.0504
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0078	5.9333	-0.0200	-0.0650	0.0339	0.1196
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0024	2.0606	-0.0061	-0.0197	0.0102	0.0504
ULS: 5b. D + 0.7E	-0.0024	2.0606	-0.0061	-0.0197	0.0102	0.0504
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0078	5.9333	-0.0200	-0.0650	0.0339	0.1196
ULS: 8. 0.6D + 0.7E	-0.0014	1.2364	-0.0036	-0.0118	0.0061	0.0302
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4153	5.9140	-0.0141	-0.0479	0.0128	16.5302
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4153	5.9140	-0.0141	-0.0479	0.0128	16.5302
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1937	-1.2080	0.0027	0.0101	0.0051	-11.2984
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0460	-0.7828	-0.0044	-0.0127	0.0151	-19.8295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0675	8.8234	-0.0261	-0.0861	0.0358	12.4795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0675	8.8234	-0.0261	-0.0861	0.0358	12.4795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8893	3.4818	-0.0135	-0.0426	0.0301	-8.3920
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7785	3.8007	-0.0188	-0.0597	0.0376	-14.7903
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0621	4.9506	-0.0121	-0.0408	0.0121	12.4103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0621	4.9506	-0.0121	-0.0408	0.0121	12.4103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8947	-0.3909	0.0005	0.0027	0.0064	-8.4612
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7839	-0.0720	-0.0048	-0.0144	0.0139	-14.8595
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4144	5.0898	-0.0117	-0.0400	0.0087	16.5101
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4144	5.0898	-0.0117	-0.0400	0.0087	16.5101
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1946	-2.0322	0.0051	0.0180	0.0010	-11.3185
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0469	-1.6071	-0.0020	-0.0048	0.0111	-19.8496

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.9460
Shear X	-2.3628
Shear Z	-0.0434
Moment X	-0.1432
Moment Y (Twist)	0.0696
Moment Z	34.4298

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	8.8234
Shear X	-1.4153
Shear Z	-0.0261
Moment X	-0.0861
Moment Y (Twist)	0.0418
Moment Z	19.8496

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	0.0023	2.1668	0.0027	0.0085	-0.0037	0.0071
ULS: 2. D + L	0.0023	2.1668	0.0027	0.0085	-0.0037	0.0071
ULS: 3. D + (S or Lr or R)	0.0094	7.6643	0.0111	0.0348	-0.0150	-0.0348
ULS: 3. D + (S or Lr or R)	0.0023	2.1668	0.0027	0.0085	-0.0037	0.0071
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0076	6.2899	0.0090	0.0282	-0.0122	-0.0243

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0023	2.1668	0.0027	0.0085	-0.0037	0.0071
ULS: 5b. D + 0.7E	0.0023	2.1668	0.0027	0.0085	-0.0037	0.0071
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0076	6.2899	0.0090	0.0282	-0.0122	-0.0243
ULS: 8. 0.6D + 0.7E	0.0014	1.3001	0.0016	0.0051	-0.0022	0.0043
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4755	6.2557	0.0155	0.0491	-0.0207	17.2005
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4755	6.2557	0.0155	0.0491	-0.0207	17.2005
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2590	-1.3035	-0.0069	-0.0218	0.0085	-11.8342
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0855	-0.8462	-0.0093	-0.0294	0.0139	-20.5890
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1007	9.3566	0.0186	0.0587	-0.0250	12.8707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1007	9.3566	0.0186	0.0587	-0.0250	12.8707
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9502	3.6872	0.0018	0.0055	-0.0031	-8.9053
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8200	4.0302	-0.0000	-0.0002	0.0010	-15.4714
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1060	5.2335	0.0123	0.0390	-0.0164	12.9022
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1060	5.2335	0.0123	0.0390	-0.0164	12.9022
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9448	-0.4359	-0.0045	-0.0142	0.0055	-8.8738
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8147	-0.0929	-0.0063	-0.0200	0.0095	-15.4399
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4764	5.3890	0.0145	0.0457	-0.0192	17.1977
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4764	5.3890	0.0145	0.0457	-0.0192	17.1977
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2581	-2.1702	-0.0080	-0.0252	0.0100	-11.8370
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0846	-1.7129	-0.0104	-0.0328	0.0154	-20.5918

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	14.8031
Shear X	-2.4630
Shear Z	0.0295
Moment X	0.0934
Moment Y (Twist)	0.0396
Moment Z	35.8234

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	9.3566
Shear X	-1.4764
Shear Z	0.0186
Moment X	0.0587
Moment Y (Twist)	0.0250
Moment Z	20.5918

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	0.0024	2.1664	-0.0027	-0.0096	0.0031	0.0063
ULS: 2. D + L	0.0024	2.1664	-0.0027	-0.0096	0.0031	0.0063
ULS: 3. D + (S or Lr or R)	0.0098	7.6623	-0.0112	-0.0394	0.0129	-0.0381
ULS: 3. D + (S or Lr or R)	0.0024	2.1664	-0.0027	-0.0096	0.0031	0.0063
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0080	6.2883	-0.0091	-0.0320	0.0104	-0.0270
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0024	2.1664	-0.0027	-0.0096	0.0031	0.0063
ULS: 5b. D + 0.7E	0.0024	2.1664	-0.0027	-0.0096	0.0031	0.0063
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0080	6.2883	-0.0091	-0.0320	0.0104	-0.0270
ULS: 8. 0.6D + 0.7E	0.0014	1.2998	-0.0016	-0.0057	0.0019	0.0038
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4751	6.2540	-0.0156	-0.0528	0.0193	17.1966
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4751	6.2540	-0.0156	-0.0528	0.0193	17.1966
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.2588	-1.3029	0.0069	0.0227	-0.0083	-11.8325
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0854	-0.8458	0.0093	0.0302	-0.0135	-20.5880

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	-1.1001	9.3541	-0.0188	-0.0644	0.0225	12.8657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1001	9.3541	-0.0188	-0.0644	0.0225	12.8657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9503	3.6864	-0.0019	-0.0077	0.0018	-8.9062
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8202	4.0292	-0.0001	-0.0022	-0.0021	-15.4728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1057	5.2321	-0.0124	-0.0420	0.0152	12.8990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1057	5.2321	-0.0124	-0.0420	0.0152	12.8990
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9447	-0.4356	0.0045	0.0147	-0.0055	-8.8728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8147	-0.0927	0.0063	0.0202	-0.0094	-15.4394
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4760	5.3875	-0.0145	-0.0490	0.0180	17.1941
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4760	5.3875	-0.0145	-0.0490	0.0180	17.1941
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.2579	-2.1694	0.0080	0.0266	-0.0096	-11.8351
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0845	-1.7123	0.0104	0.0340	-0.0148	-20.5905

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	14.7991
Shear X	-2.4624
Shear Z	-0.0296
Moment X	-0.1010
Moment Y (Twist)	0.0366
Moment Z	35.8226

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	9.3541
Shear X	-1.4760
Shear Z	-0.0188
Moment X	-0.0644
Moment Y (Twist)	0.0225
Moment Z	20.5905

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	-0.0024	2.0607	0.0061	0.0188	-0.0101	0.0505
ULS: 2. D + L	-0.0024	2.0607	0.0061	0.0188	-0.0101	0.0505
ULS: 3. D + (S or Lr or R)	-0.0096	7.2246	0.0249	0.0765	-0.0414	0.1432
ULS: 3. D + (S or Lr or R)	-0.0024	2.0607	0.0061	0.0188	-0.0101	0.0505
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0078	5.9336	0.0202	0.0621	-0.0336	0.1200
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0024	2.0607	0.0061	0.0188	-0.0101	0.0505
ULS: 5b. D + 0.7E	-0.0024	2.0607	0.0061	0.0188	-0.0101	0.0505
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0078	5.9336	0.0202	0.0621	-0.0336	0.1200
ULS: 8. 0.6D + 0.7E	-0.0014	1.2364	0.0037	0.0113	-0.0061	0.0303
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4154	5.9143	0.0142	0.0444	-0.0135	16.5309
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4154	5.9143	0.0142	0.0444	-0.0135	16.5309
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1937	-1.2081	-0.0026	-0.0089	-0.0043	-11.2983
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0460	-0.7829	0.0044	0.0136	-0.0142	-19.8321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0676	8.8238	0.0262	0.0813	-0.0361	12.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0676	8.8238	0.0262	0.0813	-0.0361	12.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8892	3.4820	0.0136	0.0412	-0.0292	-8.3916
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7784	3.8009	0.0189	0.0582	-0.0366	-14.7920
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0621	4.9509	0.0122	0.0380	-0.0126	12.4108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0621	4.9509	0.0122	0.0380	-0.0126	12.4108
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8947	-0.3909	-0.0004	-0.0020	-0.0058	-8.4611
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7839	-0.0720	0.0048	0.0149	-0.0132	-14.8615

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4144	5.0901	0.0117	0.0369	-0.0094	16.5107
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4144	5.0901	0.0117	0.0369	-0.0094	16.5107
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1947	-2.0324	-0.0051	-0.0165	-0.0003	-11.3185
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0469	-1.6072	0.0020	0.0061	-0.0102	-19.8523

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.9468
Shear X	-2.3629
Shear Z	0.0437
Moment X	0.1354
Moment Y (Twist)	0.0682
Moment Z	34.4351

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	8.8238
Shear X	-1.4154
Shear Z	0.0262
Moment X	0.0813
Moment Y (Twist)	0.0414
Moment Z	19.8523

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: Norris Pool House - 4x11 - 60INsquare - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t_w (in)				
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				

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

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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{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	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.11,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04,1.04,1.0	300	200	1
114	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.04,1.04,1.04,2.19,1.04,1.04,1.04,1.04,1.04,1.04,1.0	300	200	1
115	19	10.20	10.20	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.15,1.14,1.15,1.15,1.15,1.15,1.15,1.16,1.16,1.16,1.16,1.15,1.15,1.1	300	200	1
116	19	10.20	10.20	10.20	1.16,1.1	300	200	1
201	7	22.20	22.20	10.57	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
204	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
208	19	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.08,2.08,2.08,2.06,2.08,2.08,2.08,1.83,2.08,2.08,2.07,2.07,2.08,2.08,2.0	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
211	19	1.33	1.33	2.05	2.08,2.08,2.08,2.08,2.08,2.08,2.07,2.07,1.65,1.91,2.06,2.06,2.06,2.05,2.07,2.07,2.09,2.11,2.07,2.07,1.8	300	200	1
212	5	4.20	4.20	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.11,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.04,1.04,1.04,1.0	300	200	1
214	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.14,1.04,1.04,1.04,2.19,1.04,1.04,1.04,1.04,1.04,1.04,1.0	300	200	1
215	19	10.20	10.20	10.20	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.12,1.10,1.11,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.13,1.12,1.12,1.1	300	200	1
216	19	10.20	10.20	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.1	300	200	1
301	7	22.20	22.20	10.57	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
304	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
305	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
306	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
307	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.64,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
308	19	9.97	9.97	4.75	2.33,2.3	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.70,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
311	19	9.97	9.97	4.75	2.33,2.3	300	200	1
312	5	4.20	4.20	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.09,1.08,1.12,1.09,1.09,1.09,1.10,1.09,1.09,1.10,1.09,1.09,1.09,1.0	300	200	1
314	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.16,1.09,1.09,1.09,1.59,1.09,1.09,1.09,1.10,1.09,1.09,1.1	300	200	1

314	19	4.88	4.00	0	0,1.18,1.09,1.09,1.09,2.91	0	0	1
315	19	10.2 0	10.2 0	10.2 20	1.22,1.22,1.22,1.22,1.22,1.22,1.23,1.23,1.24,1.24,1.23,1.23,1.23,1.22,1.22,1.22,1.21,1.23,1.23,1.24,1.24,1.23,1.23,1.23,1.23	30 0	20 0	1
316	19	10.2 0	10.2 0	10.2 20	1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.22,1.24,1.22,1.22,1.22,1.28,1.22,1.22,1.22,1.23,1.22,1.22,1.22,1.24,1.22,1.22,1.22,1.33	30 0	20 0	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	89.59	42.30	42.30	75.35	75.35
2	198.33	182.14	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	24.66	6.12	40.24	43.62
14	133.20	85.85	25.05	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62
101	251.16	89.59	42.30	42.30	75.35	75.35
102	198.33	182.14	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	23.74	6.12	40.24	43.62
114	133.20	85.85	23.80	6.12	40.24	43.62
115	133.20	31.52	17.59	6.12	40.24	43.62
116	133.20	31.52	17.74	6.12	40.24	43.62
201	251.16	89.59	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	182.14	21.95	21.95	59.50	59.50
213	133.20	85.85	23.75	6.12	40.24	43.62
214	133.20	85.85	23.80	6.12	40.24	43.62
215	133.20	31.52	17.07	6.12	40.24	43.62
216	133.20	31.52	17.34	6.12	40.24	43.62
301	251.16	89.59	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	32.95	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	32.95	32.87	6.12	40.24	43.62
312	198.33	182.14	21.95	21.95	59.50	59.50
313	133.20	85.85	24.66	6.12	40.24	43.62
314	133.20	85.85	25.06	6.12	40.24	43.62
315	133.20	31.52	18.79	6.12	40.24	43.62
316	133.20	31.52	18.87	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	0.156	0.814	0.007	0.031	0.001	0.817	#16	0.593	Not Required	Pass
2	0.003	0.552	0.118	0.115	0.020	0.650	#21	0.114	Not Required	Pass
3	0.007	0.798	0.041	0.080	0.004	0.843	#21	0.045	Not Required	Pass
4	0.008	0.774	0.130	0.078	0.027	0.856	#21	0.080	Not Required	Pass
5	0.007	0.495	0.138	0.079	0.035	0.529	#21	0.074	Not Required	Pass
6	0.008	0.795	0.045	0.079	0.005	0.843	#21	0.045	Not Required	Pass
7	0.008	0.494	0.143	0.079	0.036	0.533	#21	0.074	Not Required	Pass
8	0.001	0.058	0.125	0.054	0.013	0.170	#21	0.095	Not Required	Pass
9	0.013	0.092	0.040	0.001	0.000	0.136	#21	0.204	Not Required	Pass
10	0.008	0.767	0.140	0.077	0.030	0.864	#21	0.080	Not Required	Pass
11	0.000	0.059	0.127	0.056	0.013	0.174	#21	0.095	Not Required	Pass
12	0.003	0.537	0.117	0.115	0.020	0.632	#21	0.035	Not Required	Pass
13	0.006	0.315	0.327	0.071	0.017	0.629	#21	0.286	Not Required	Pass
14	0.007	0.312	0.327	0.069	0.017	0.621	#21	0.190	Not Required	Pass
15	0.000	0.123	0.175	0.042	0.010	0.298	#21	Not Required	Not Required	Pass
16	0.000	0.120	0.175	0.041	0.010	0.294	#21	Not Required	Not Required	Pass
101	0.165	0.847	0.005	0.033	0.000	0.850	#16	0.593	Not Required	Pass
102	0.002	0.566	0.120	0.122	0.020	0.663	#21	0.114	Not Required	Pass
103	0.008	0.840	0.049	0.084	0.007	0.894	#21	0.045	Not Required	Pass
104	0.008	0.815	0.131	0.082	0.028	0.909	#21	0.080	Not Required	Pass
105	0.008	0.522	0.134	0.083	0.034	0.557	#21	0.074	Not Required	Pass
106	0.008	0.851	0.051	0.085	0.008	0.905	#21	0.045	Not Required	Pass
107	0.008	0.528	0.134	0.084	0.034	0.564	#21	0.074	Not Required	Pass
108	0.001	0.071	0.117	0.053	0.013	0.157	#21	0.095	Not Required	Pass
109	0.010	0.082	0.038	0.001	0.000	0.126	#21	0.204	Not Required	Pass
110	0.008	0.824	0.128	0.083	0.027	0.912	#21	0.080	Not Required	Pass

110	0.000	0.024	0.120	0.003	0.027	0.912	#21	0.000	Not Required	Pass
111	0.001	0.073	0.119	0.054	0.013	0.160	#21	0.095	Not Required	Pass
112	0.002	0.576	0.122	0.123	0.021	0.674	#21	0.035	Not Required	Pass
113	0.006	0.262	0.312	0.070	0.017	0.550	#21	0.286	Not Required	Pass
114	0.007	0.263	0.310	0.068	0.017	0.544	#21	0.286	Not Required	Pass
115	0.001	0.280	0.169	0.053	0.013	0.448	#21	0.728	Not Required	Pass
116	0.001	0.272	0.170	0.052	0.013	0.441	#21	0.728	Not Required	Pass
201	0.165	0.847	0.005	0.033	0.000	0.850	#16	0.593	Not Required	Pass
202	0.002	0.575	0.122	0.123	0.021	0.673	#21	0.035	Not Required	Pass
203	0.008	0.850	0.051	0.085	0.008	0.905	#21	0.045	Not Required	Pass
204	0.008	0.823	0.128	0.083	0.027	0.912	#21	0.080	Not Required	Pass
205	0.008	0.528	0.134	0.084	0.034	0.564	#21	0.074	Not Required	Pass
206	0.008	0.840	0.049	0.084	0.007	0.894	#21	0.045	Not Required	Pass
207	0.008	0.522	0.134	0.083	0.034	0.557	#21	0.074	Not Required	Pass
208	0.001	0.067	0.120	0.052	0.013	0.161	#21	0.095	Not Required	Pass
209	0.010	0.082	0.038	0.001	0.000	0.125	#21	0.204	Not Required	Pass
210	0.008	0.815	0.131	0.082	0.028	0.908	#21	0.080	Not Required	Pass
211	0.000	0.070	0.121	0.053	0.013	0.163	#21	0.095	Not Required	Pass
212	0.002	0.566	0.120	0.122	0.020	0.662	#21	0.114	Not Required	Pass
213	0.006	0.262	0.312	0.070	0.017	0.549	#21	0.286	Not Required	Pass
214	0.007	0.262	0.310	0.068	0.017	0.544	#21	0.286	Not Required	Pass
215	0.002	0.296	0.161	0.054	0.013	0.457	#21	0.728	Not Required	Pass
216	0.001	0.287	0.163	0.053	0.013	0.450	#21	0.728	Not Required	Pass
301	0.156	0.814	0.008	0.031	0.001	0.817	#16	0.593	Not Required	Pass
302	0.003	0.537	0.117	0.115	0.020	0.632	#21	0.035	Not Required	Pass
303	0.008	0.795	0.045	0.079	0.005	0.843	#21	0.045	Not Required	Pass
304	0.008	0.767	0.140	0.077	0.030	0.865	#21	0.080	Not Required	Pass
305	0.008	0.495	0.143	0.079	0.037	0.533	#21	0.074	Not Required	Pass
306	0.007	0.799	0.041	0.080	0.004	0.844	#21	0.045	Not Required	Pass
307	0.007	0.495	0.138	0.079	0.035	0.530	#21	0.074	Not Required	Pass
308	0.000	0.120	0.175	0.041	0.010	0.294	#21	Not Required	Not Required	Pass
309	0.013	0.092	0.040	0.001	0.000	0.136	#21	0.204	Not Required	Pass
310	0.008	0.774	0.130	0.078	0.027	0.856	#21	0.080	Not Required	Pass
311	0.000	0.123	0.175	0.042	0.010	0.298	#21	Not Required	Not Required	Pass
312	0.003	0.552	0.118	0.116	0.020	0.650	#21	0.114	Not Required	Pass
313	0.006	0.315	0.327	0.071	0.017	0.629	#21	0.190	Not Required	Pass
314	0.007	0.312	0.327	0.069	0.017	0.621	#21	0.286	Not Required	Pass
315	0.002	0.268	0.172	0.056	0.013	0.440	#21	0.728	Not Required	Pass
316	0.001	0.262	0.173	0.054	0.013	0.435	#21	0.728	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS
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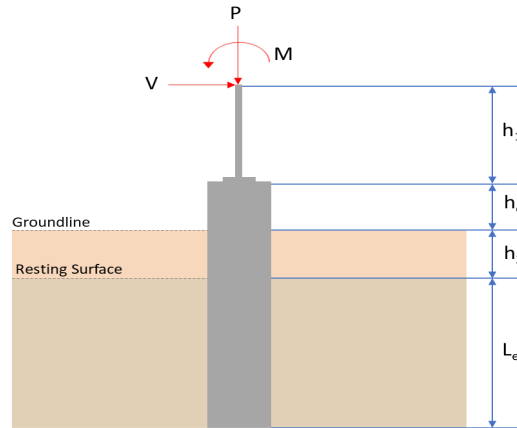
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 5.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.823	13.946
V_x (kip)	-1.415	-2.363
V_z (kip)	-0.026	-0.043
M_x (kipft)	-0.086	-0.143
M_z (kipft)	19.850	34.430

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.415 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.18025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.85 \text{ kipft}) + ((-1.415 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.5287 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.2587 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.026 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0033121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.086 \text{ kipft}) + ((-0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.010955 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.88808 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.2587 \text{ ft}), (0.88808 \text{ ft})]$$

$$L_{e,req} = 5.259 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.259 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.95618$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.823 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.35292 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35292 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17646$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.1$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18025 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.5287 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5287 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.5287 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7616 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.5287 \text{ kipft/ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.5287 \text{ kipft/ft})) + (2 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.22561 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.5287 \text{ kipft/ft})) + ((-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.80646 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7616 \text{ ft})}{2}$$

$$p_a = 0.28212 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22561 \text{ kip/ft}^2)}{(0.28212 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79967$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.80646 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97753$$

Status: **PASS**
Ratio: **0.800**

Status: **PASS**
Ratio: **0.980**

Considering z-direction:

$H_o = -0.0033121 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010955 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010955 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.010955 \text{ kipft/ft})) + (4 \times (-0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9076 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010955 \text{ kipft/ft})) + (3 \times (-0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.010955 \text{ kipft/ft})) + (2 \times (-0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.00081498 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010955 \text{ kipft/ft})) + ((-0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.00073275 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9076 \text{ ft})}{2}$$

$$p_a = 0.29307 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00081498 \text{ kip/ft}^2)}{(0.29307 \text{ kip/ft}^2)}$$

$$Ratio = -0.0027808$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

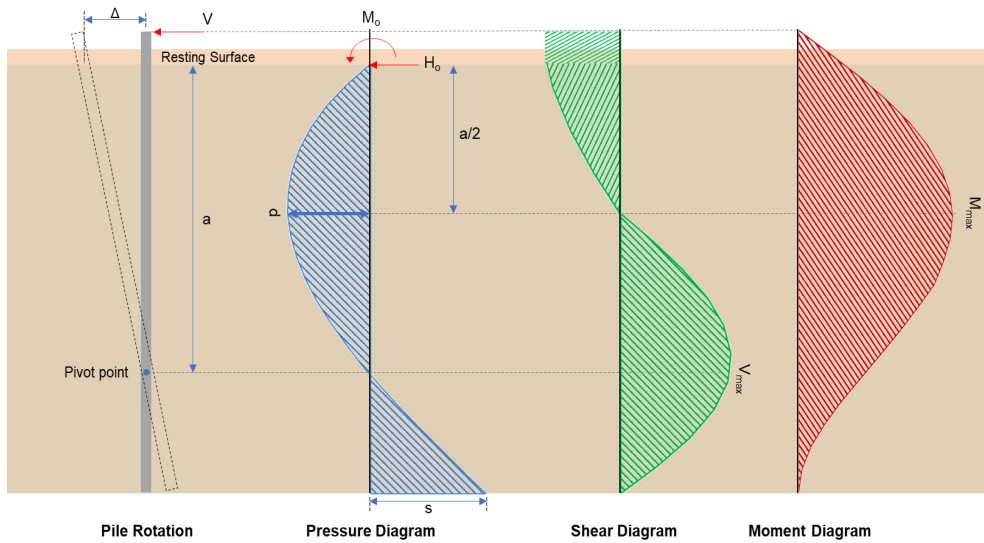
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00073275 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.00088818$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.363 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.30102 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.43 \text{ kipft}) + ((-2.363 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.386 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.386 \text{ kipft/ft})}{(-0.30102 \text{ kip/ft})}$$

$$E = 14.57 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.386 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times 4.386) + (4 \times (-0.30102) \times 5.5)}$$

$$a = \frac{(6 \times (4.386 \text{ kipft/ft})) + (4 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft}))}{(6 \times (4.386 \text{ kipft/ft})) + (4 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7588 \text{ ft}$$

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

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

$$V_{max} = ((-0.30102 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.57 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7588 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.57 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7588 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0531 \text{ kip}$$

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

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

$$M_{max} = ((-0.30102 \text{ kip/ft}) \times (60 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(14.57 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7588 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.57 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7588 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.57 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7588 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.39 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.043 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0054777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.143 \text{ kipft}) + ((-0.043 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.018217 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.018217 \text{ kipft/ft})}{(-0.0054777 \text{ kip/ft})}$$

$$E = 3.3256 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.018217 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.0054777 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.018217 \text{ kipft/ft})) + (4 \times (-0.0054777 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.907 \text{ ft}$$

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

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

$$V_{max} = ((-0.0054777 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3256 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.907 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.3256 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.907 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.047501 \text{ kip}$$

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

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

$$M_{max} = ((-0.0054777 \text{ kip/ft}) \times (60 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(3.3256 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.907 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.3256 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.907 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (3.3256 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.907 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 0.11716 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.946 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.72 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.72 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96007$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.946 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0033355$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.946 \text{ kip} \rightarrow 13946 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(13946 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.98 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.98 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.98 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.98 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.49 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.0531 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.0531 \text{ kip})}{(152.49 \text{ kip})}$$

$$Ratio = 0.052811$$

Considering z-direction:

$V_{max} = 0.047501 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.047501 \text{ kip})}{(152.49 \text{ kip})}$$

$$Ratio = 0.00031151$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 21.39 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.39 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.043878$$

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

Considering z-direction:

$M_{max} = 0.11716 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.11716 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00024032$$

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

REFERENCES	CALCULATIONS	RESULTS
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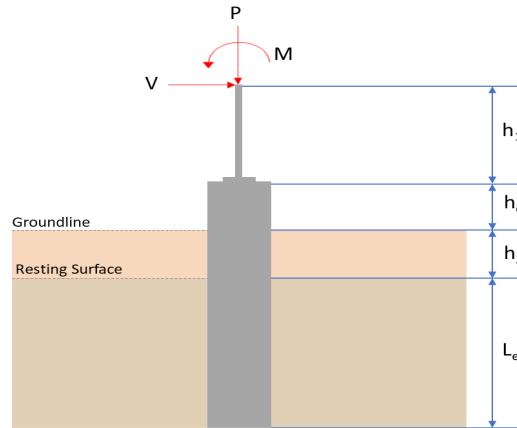
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 5.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.824	13.947
V_x (kip)	-1.415	-2.363
V_z (kip)	0.026	0.044
M_x (kipft)	0.081	0.135
M_z (kipft)	19.852	34.435

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.415 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.18025 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.852 \text{ kipft}) + ((-1.415 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.5289 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.2589 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0033121 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.081 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.010318 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.0086 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.2589 \text{ ft}), (1.0086 \text{ ft})]$$

$$L_{e,req} = 5.259 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.259 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.95618$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.824 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.35296 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35296 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17648$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.1$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18025 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.5289 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5289 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.5289 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7616 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.5289 \text{ kipft/ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.5289 \text{ kipft/ft})) + (2 \times (-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.22564 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.5289 \text{ kipft/ft})) + ((-0.18025 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.80657 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7616 \text{ ft})}{2}$$

$$p_a = 0.28212 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22564 \text{ kip/ft}^2)}{(0.28212 \text{ kip/ft}^2)}$$

$$Ratio = 0.7998$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.80657 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.97765$$

Status: **PASS**
Ratio: **0.800**

Status: **PASS**
Ratio: **0.980**

Considering z-direction:

$H_o = 0.0033121 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010318 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010318 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.010318 \text{ kipft/ft})) + (4 \times (0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9145 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010318 \text{ kipft/ft})) + (3 \times (0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.010318 \text{ kipft/ft})) + (2 \times (0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.0033853 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010318 \text{ kipft/ft})) + ((0.0033121 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.0077065 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9145 \text{ ft})}{2}$$

$$p_a = 0.29358 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0033853 \text{ kip/ft}^2)}{(0.29358 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.011531$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

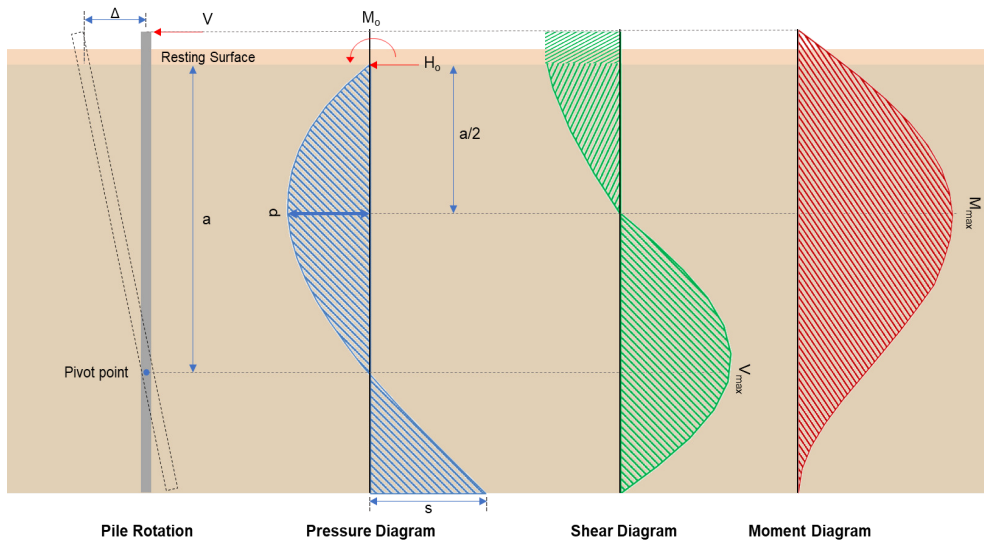
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.0077065 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0093412$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.363 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.30102 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.435 \text{ kipft}) + ((-2.363 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.3866 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3866 \text{ kipft/ft})}{(-0.30102 \text{ kip/ft})}$$

$$E = 14.573 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3866 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.3866 \text{ kipft/ft})) + (4 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = \frac{(6 \times (4.3866 \text{ kipft/ft})) + (4 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft}))}{(6 \times (4.3866 \text{ kipft/ft})) + (4 \times (-0.30102 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7588 \text{ ft}$$

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

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

$$V_{max} = ((-0.30102 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.573 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7588 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.573 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7588 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0541 \text{ kip}$$

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

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

$$M_{max} = ((-0.30102 \text{ kip/ft}) \times (60 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(14.573 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7588 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.573 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7588 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (14.573 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7588 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 21.393 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.044 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0056051 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.135 \text{ kipft}) + ((0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.017197 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017197 \text{ kipft/ft})}{(0.0056051 \text{ kip/ft})}$$

$$E = 3.0682 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017197 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.0056051 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.017197 \text{ kipft/ft})) + (4 \times (0.0056051 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9162 \text{ ft}$$

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

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

$$V_{max} = ((0.0056051 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.0682 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9162 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.0682 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9162 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.046306 \text{ kip}$$

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

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

$$M_{max} = ((0.0056051 \text{ kip/ft}) \times (60 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(3.0682 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9162 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.0682 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9162 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.0682 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9162 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.11357 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.947 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.72 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.72 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96007$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.947 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0033357$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.947 \text{ kip} \rightarrow 13947 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(13947 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.98 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (170.98 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 170.98 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.98 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.49 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.0541 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(8.0541 \text{ kip})}{(152.49 \text{ kip})}$$

$$Ratio = 0.052818$$

Considering z-direction:

$V_{max} = 0.046306 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.046306 \text{ kip})}{(152.49 \text{ kip})}$$

$$Ratio = 0.00030367$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 21.393 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.393 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.043884$$

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

Considering z-direction:

$M_{max} = 0.11357 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.11357 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.00023297$$

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

REFERENCES	CALCULATIONS	RESULTS
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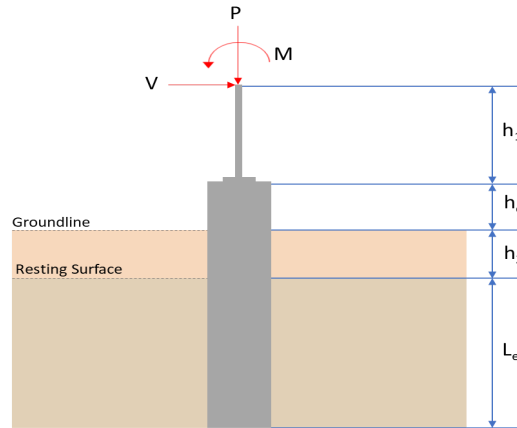
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.357	14.803
V_x (kip)	-1.476	-2.463
V_z (kip)	0.019	0.029
M_x (kipft)	0.059	0.093
M_z (kipft)	20.592	35.823

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.476 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.18803 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.592 \text{ kipft}) + ((-1.476 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.6232 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3124 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.019 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0024204 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.059 \text{ kipft}) + ((0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0075159 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.90143 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.3124 \text{ ft}), (0.90143 \text{ ft})]$$

$$L_{e,req} = 5.312 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.312 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92383$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.357 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.37428 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37428 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18714$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.15$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18803 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6232 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6232 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.6232 \text{ kipft/ft})) + (4 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9366 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6232 \text{ kipft/ft})) + (3 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.6232 \text{ kipft/ft})) + (2 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20888 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6232 \text{ kipft/ft})) + ((-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.75588 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9366 \text{ ft})}{2}$$

$$p_a = 0.29525 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20888 \text{ kip/ft}^2)}{(0.29525 \text{ kip/ft}^2)}$$

$$Ratio = 0.70747$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.75588 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.87638$$

Status: **PASS**
Ratio: **0.710**

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = 0.0024204 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0075159 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0075159 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.0075159 \text{ kipft/ft})) + (4 \times (0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0981 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0075159 \text{ kipft/ft})) + (3 \times (0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.0075159 \text{ kipft/ft})) + (2 \times (0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.0023221 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0075159 \text{ kipft/ft})) + ((0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.0052535 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0981 \text{ ft})}{2}$$

$$p_a = 0.30735 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0023221 \text{ kip/ft}^2)}{(0.30735 \text{ kip/ft}^2)}$$

$$Ratio = 0.0075552$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

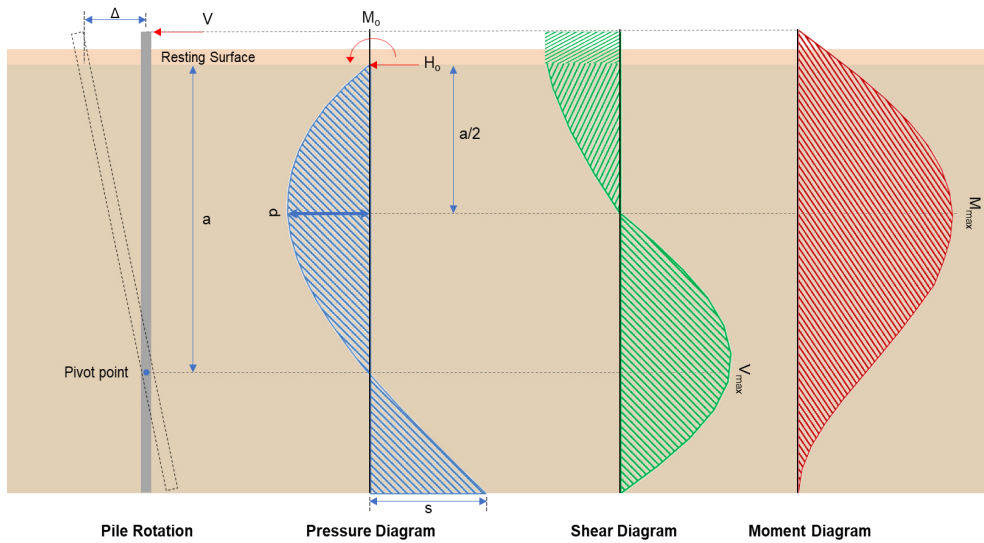
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0052535 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.006091$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.463 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.31376 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.823 \text{ kipft}) + ((-2.463 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.5634 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.5634 \text{ kipft/ft})}{(-0.31376 \text{ kip/ft})}$$

$$E = 14.544 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.5634 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.31376 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.5634 \text{ kipft/ft})) + (4 \times (-0.31376 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.31376 \text{ kip/ft}) \times (60 \text{ in})}{(6 \times (4.5634 \text{ kipft/ft})) + (4 \times (-0.31376 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9333 \text{ ft}$$

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

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

$$V_{max} = ((-0.31376 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.544 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9333 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.544 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9333 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0607 \text{ kip}$$

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

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

$$M_{max} = ((-0.31376 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(14.544 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9333 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.544 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9333 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.544 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9333 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.352 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.029 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = 0.0036943 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.093 \text{ kipft}) + ((0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.011847 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.011847 \text{ kipft/ft})}{(0.0036943 \text{ kip/ft})}$$

$$E = 3.2069 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011847 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0036943 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.011847 \text{ kipft/ft})) + (4 \times (0.0036943 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0942 \text{ ft}$$

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

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

$$V_{max} = ((0.0036943 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0942 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0942 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.030516 \text{ kip}$$

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

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

$$M_{max} = ((0.0036943 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(3.2069 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0942 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2069 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0942 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2069 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0942 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.078246 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.803 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.69 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.69 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96007$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(14.803 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0035404$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.803 \text{ kip} \rightarrow 14803 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(14803 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.09 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (171.09 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.09 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((171.09 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.56 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.0607 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.0607 \text{ kip})}{(152.56 \text{ kip})}$$

$$Ratio = 0.052836$$

Considering z-direction:

$V_{max} = 0.030516 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.030516 \text{ kip})}{(152.56 \text{ kip})}$$

$$Ratio = 0.00020002$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 22.352 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(22.352 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.045849$$

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

Considering z-direction:

$M_{max} = 0.078246 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.078246 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.0001605$$

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

REFERENCES	CALCULATIONS	RESULTS
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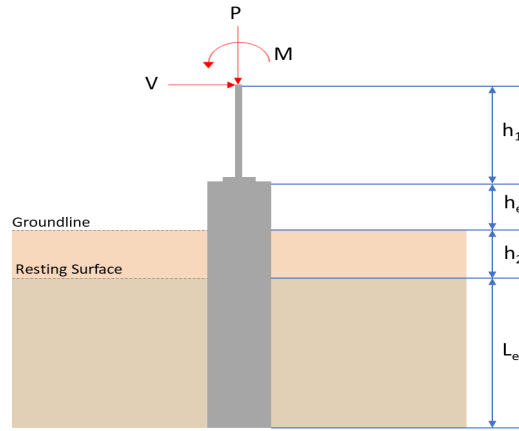
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 60$ in - Pile width

$D = 60$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.354	14.799
V_x (kip)	-1.476	-2.462
V_z (kip)	-0.019	-0.030
M_x (kipft)	-0.064	-0.101
M_z (kipft)	20.591	35.823

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.476 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.18803 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.591 \text{ kipft}) + ((-1.476 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 2.6231 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3123 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.019 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0024204 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((-0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.0081529 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.81139 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.3123 \text{ ft}), (0.81139 \text{ ft})]$$

$$L_{e,req} = 5.312 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.312 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92383$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (60 \text{ in}) \times (60 \text{ in})$$

$$A = 25 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.354 \text{ kip})}{(25 \text{ ft}^2)}$$

$$q = 0.37416 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37416 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18708$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(60 \text{ in})}$$

$$L/D = 1.15$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18803 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6231 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6231 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.6231 \text{ kipft/ft})) + (4 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9366 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6231 \text{ kipft/ft})) + (3 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.6231 \text{ kipft/ft})) + (2 \times (-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20886 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6231 \text{ kipft/ft})) + ((-0.18803 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.75584 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9366 \text{ ft})}{2}$$

$$p_a = 0.29525 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20886 \text{ kip/ft}^2)}{(0.29525 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.70741$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.75584 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87633$$

Status: **PASS**
Ratio: **0.710**

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = -0.0024204 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0081529 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0081529 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.0081529 \text{ kipft/ft})) + (4 \times (-0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0884 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0081529 \text{ kipft/ft})) + (3 \times (-0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.0081529 \text{ kipft/ft})) + (2 \times (-0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = -0.00056138 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0081529 \text{ kipft/ft})) + ((-0.0024204 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.00043346 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0884 \text{ ft})}{2}$$

$$p_a = 0.30663 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00056138 \text{ kip/ft}^2)}{(0.30663 \text{ kip/ft}^2)}$$

$$Ratio = -0.0018308$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

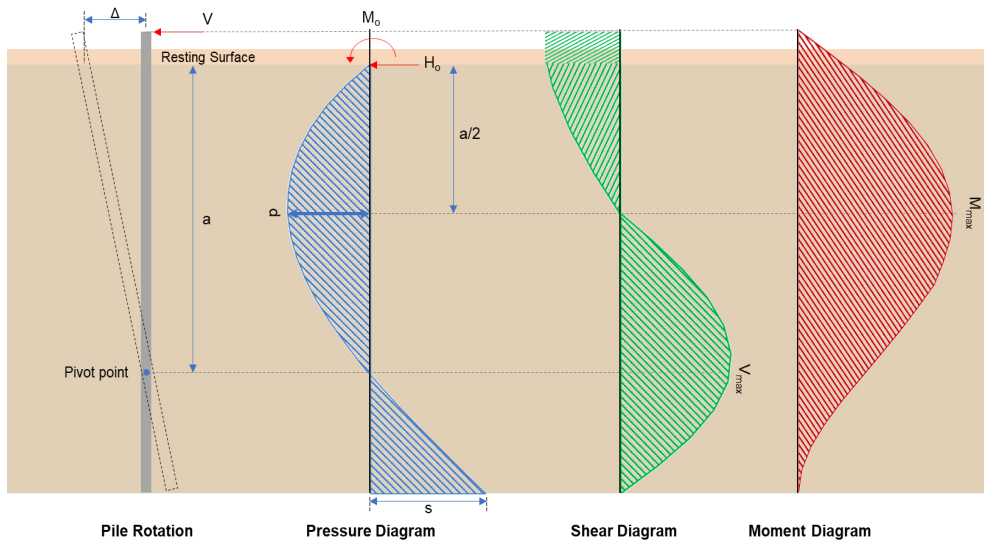
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00043346 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.00050256$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.462 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.31363 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.823 \text{ kipft}) + ((-2.462 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 4.5634 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.5634 \text{ kipft/ft})}{(-0.31363 \text{ kip/ft})}$$

$$E = 14.55 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.5634 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.31363 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.5634 \text{ kipft/ft})) + (4 \times (-0.31363 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.31363 \text{ kip/ft}) \times (60 \text{ in})}{(6 \times (4.5634 \text{ kipft/ft})) + (4 \times (-0.31363 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9332 \text{ ft}$$

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

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

$$V_{max} = ((-0.31363 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.55 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9332 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (14.55 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9332 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0603 \text{ kip}$$

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

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

$$M_{max} = ((-0.31363 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(14.55 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9332 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.55 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9332 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.55 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9332 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.351 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.03 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.101 \text{ kipft}) + ((-0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (60 \text{ in})}$$

$$M_o = 0.012866 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.012866 \text{ kipft/ft})}{(-0.0038217 \text{ kip/ft})}$$

$$E = 3.3667 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.012866 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.012866 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0884 \text{ ft}$$

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

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

$$V_{max} = ((-0.0038217 \text{ kip/ft}) \times (60 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3667 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0884 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3667 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0884 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.032499 \text{ kip}$$

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

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

$$M_{max} = ((-0.0038217 \text{ kip/ft}) \times (60 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(3.3667 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0884 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3667 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.0884 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3667 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.0884 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.083612 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.799 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2)) \right]$$

$$A_{st,required} = -131.69 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-131.69 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96007$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((60 \text{ in}), (60 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 22 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.960</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(3600 \text{ in}^2) - (6.7495 \text{ in}^2)]) + ((60 \text{ ksi}) \times (6.7495 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 4181.1 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(14.799 \text{ kip})}{(4181.1 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0035395$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 60 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (60 \text{ in})$</p> <p style="text-align: center;">$d = 48 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(48 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.58722$</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;">$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$</p>	

$$V_{c,max} = 422.8 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.799 \text{ kip} \rightarrow 14799 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + \frac{(14799 \text{ lbf})}{6 \times (3600 \text{ in}^2)}} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 171.09 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(422.8 \text{ kip}), (171.09 \text{ kip}), (529.12 \text{ kip})]$$

$$V_c = 171.09 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(1152 \text{ kip}), (63.617 \text{ kip})]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((171.09 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152.56 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.0603 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.0603 \text{ kip})}{(152.56 \text{ kip})}$$

$$Ratio = 0.052833$$

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

Considering z-direction:

$V_{max} = 0.032499 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.032499 \text{ kip})}{(152.56 \text{ kip})}$$

$$Ratio = 0.00021302$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(60 \text{ in}) \times (60 \text{ in})^2}{6}$$

$$S_m = 36000 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 35999.999 \text{ in}^3$$

$$\phi M_{n,1} = 487.500 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (36000 \text{ in}^3)$$

$$\phi M_{n,2} = 4143.7 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(487.5 \text{ kipft}), (4143.7 \text{ kipft})]$$

$$\phi M_n = 487.5 \text{ kipft}$$

Considering x-direction:

$M_{max} = 22.351 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(22.351 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$Ratio = 0.045848$$

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

Considering z-direction:

$M_{max} = 0.083612 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.083612 \text{ kipft})}{(487.5 \text{ kipft})}$$

$$\text{Ratio} = 0.00017151$$

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