Your Project Calculations

Project Name: W11232. Revised2

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=W11232.%20Revised2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=CsuxbxDtrpHoKdqdtxIb0LNM11bjp6N23xnNwJTbmHKHS1wgsLOIF0ICdOGAqfc0

Array Specification

Product:	Beam
Unique ID:	4P-19.75-6TOP-XD-45-L-4Hx10W-BK3K
Duty Classification:	XD
Module Width:	44.70 in
Module Length:	89.80in
Number of Rows:	4
Number of Columns:	10
Total Number of Modules:	40
Desired Tilt Angle:	5
Front Edge Clearance:	9
Total Array Height at Tilt:	10.31 ft
Total Frame Length:	74.25 ft
Frame Weight:	3605 lbs
Array Dimensions N/S:	15.07 ft
Array Dimensions E/W:	75.67 ft
Rail Length:	180.80 in
Rail Spacing:	3.78 ft
Rail Check:	PASS (26% utilized)

Support Specifications

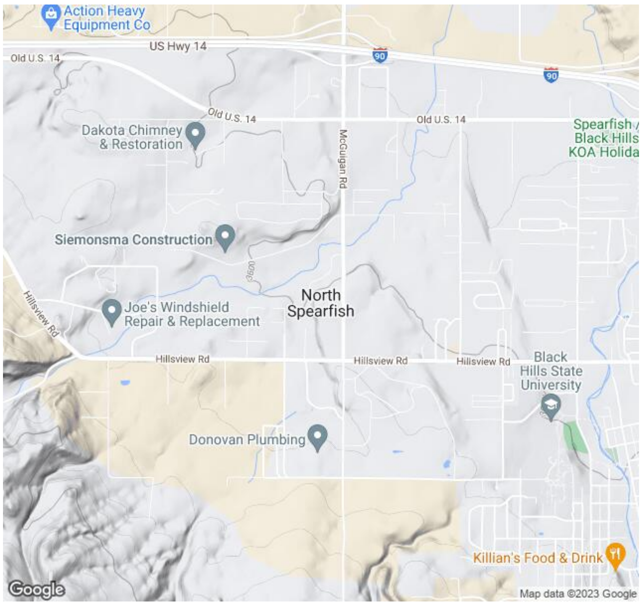
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.66 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.25 ft
Foundation Volume:	6.545 y^3
Foundation Result:	PASSED
Mount Twist:	0.015411 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	F3J9+J6 North Spearfish, SD, USA
Wind Speed:	115 mph
Snow Load:	43 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.026006 ksf



AutoDesigner Input

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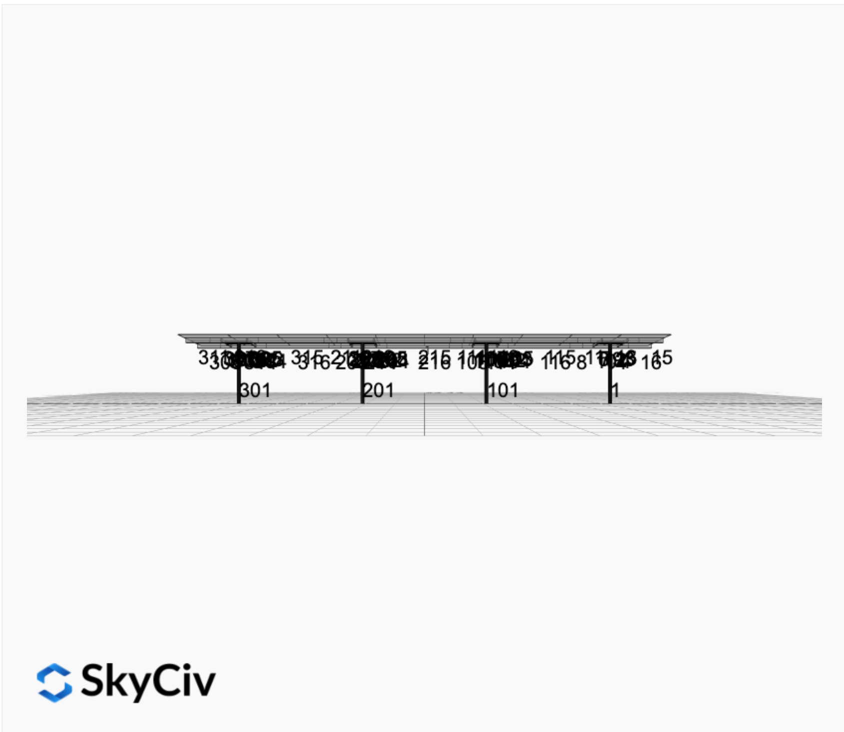
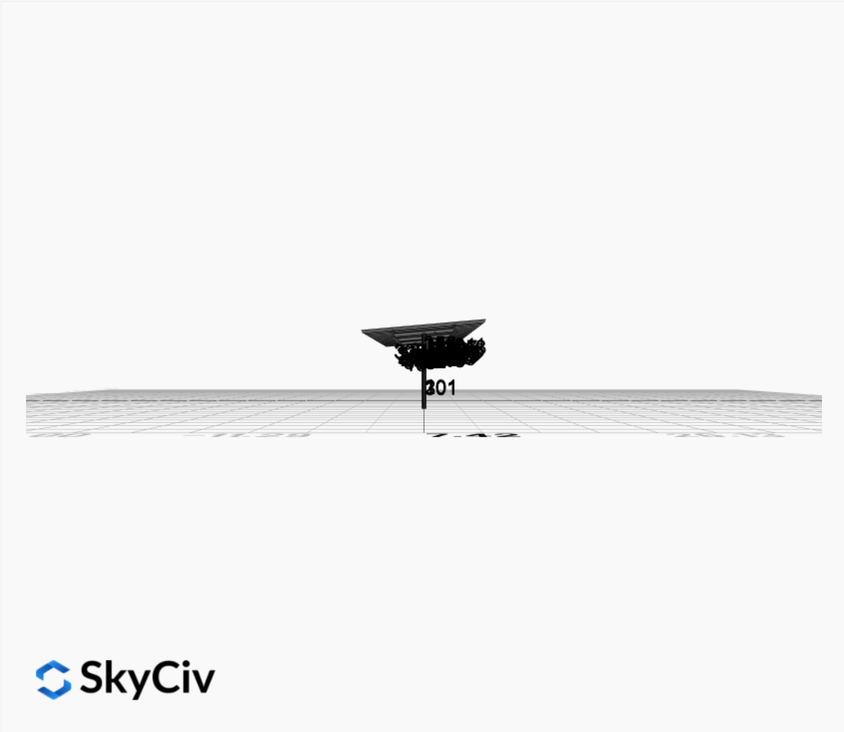
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent

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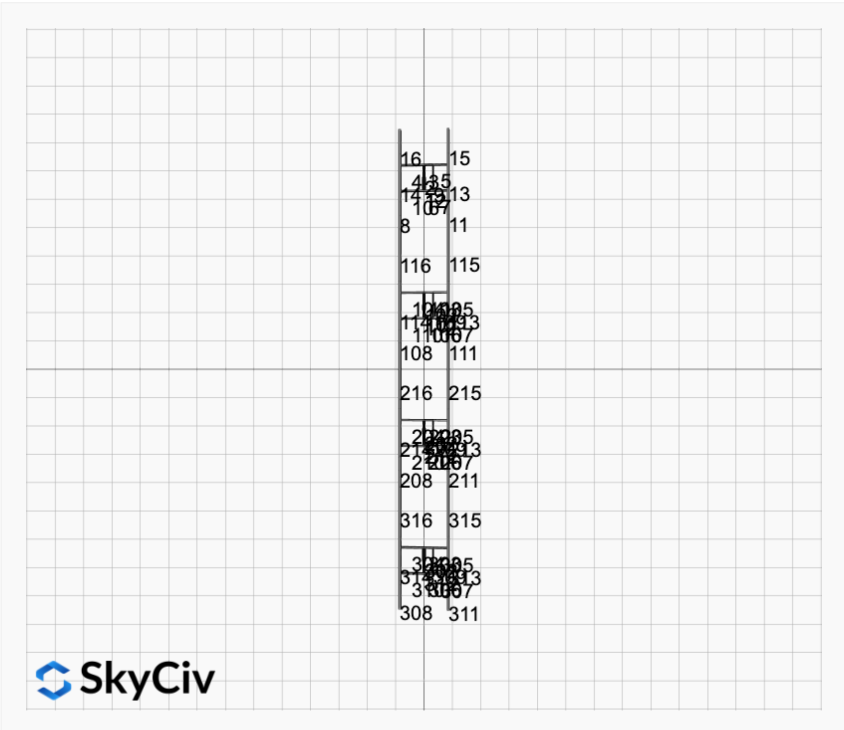
SkyCiv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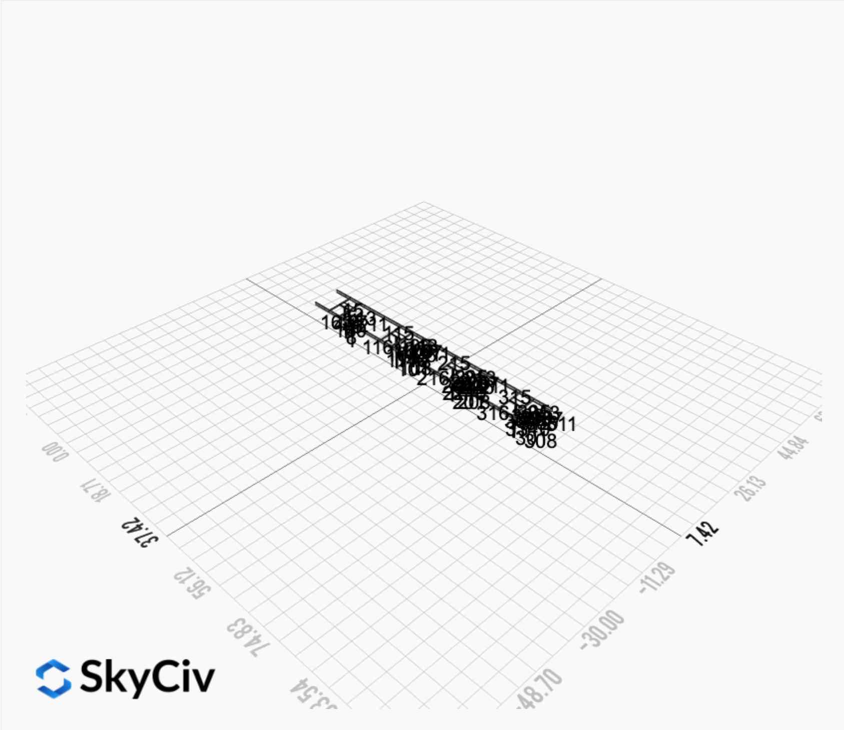
SkyCiv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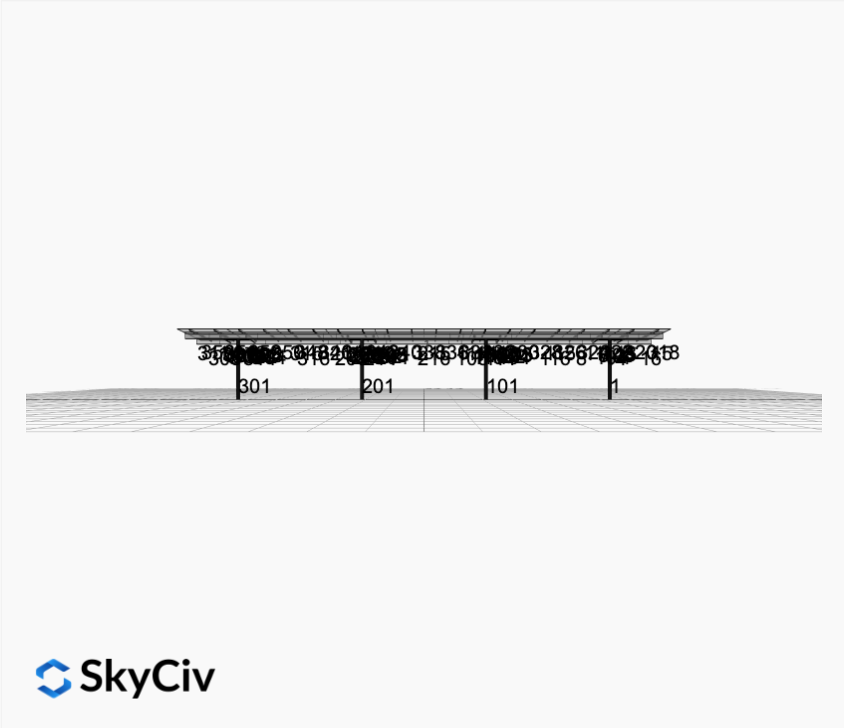
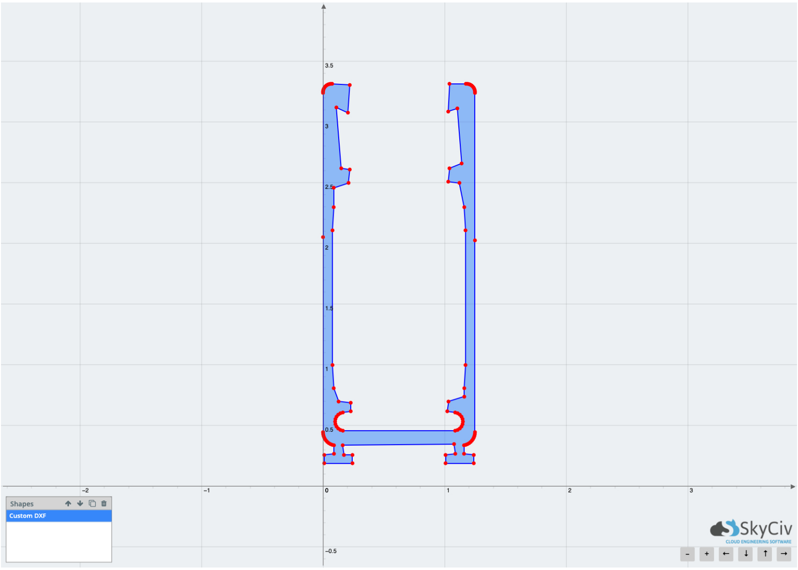
37.42
56.12
74.83
7.42
-11.29
-30.00
-48.70
26.13
44.84

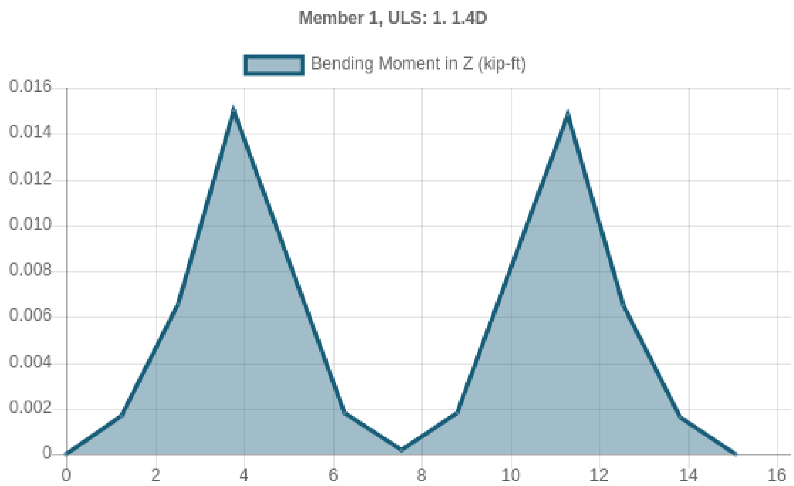

301
201
101
1

Rail Design Check

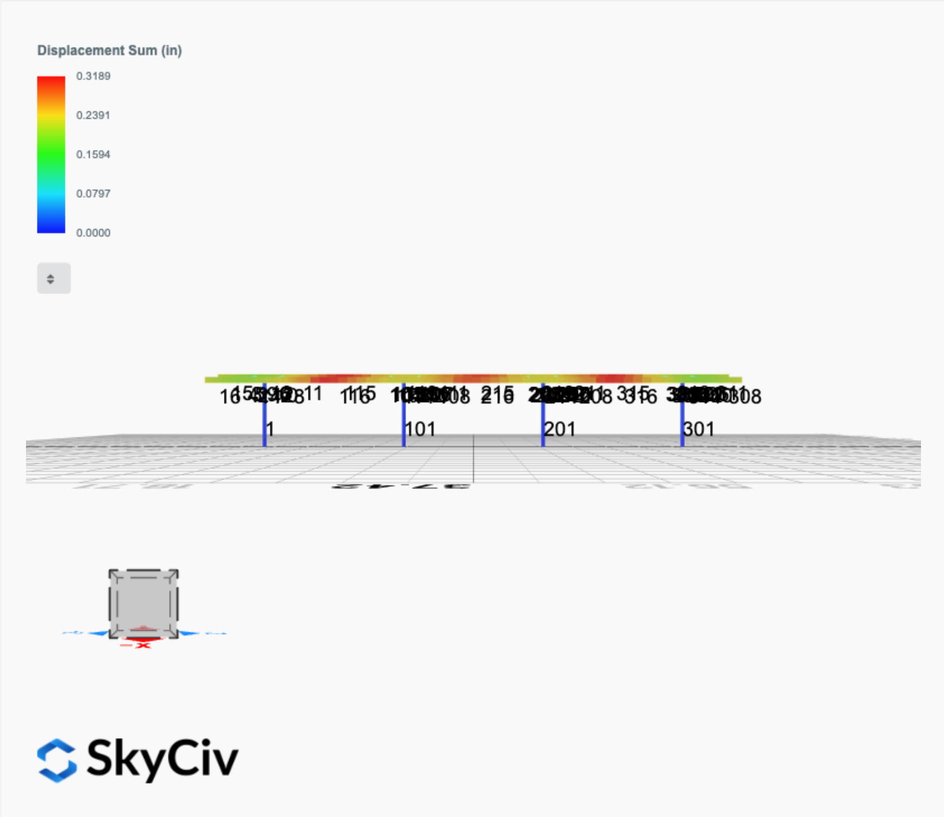
Rail Length: 15.066666666666668 ft
Additional Restraints Required: None
Tributary Width: 3.7833333333333333 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0980 kip/ft
Snow (Y): -0.0086 kip/ft
Wind uplift Case A: 0.0259 kip/ft
Wind uplift Case A: 0.0259 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0594 kip/ft

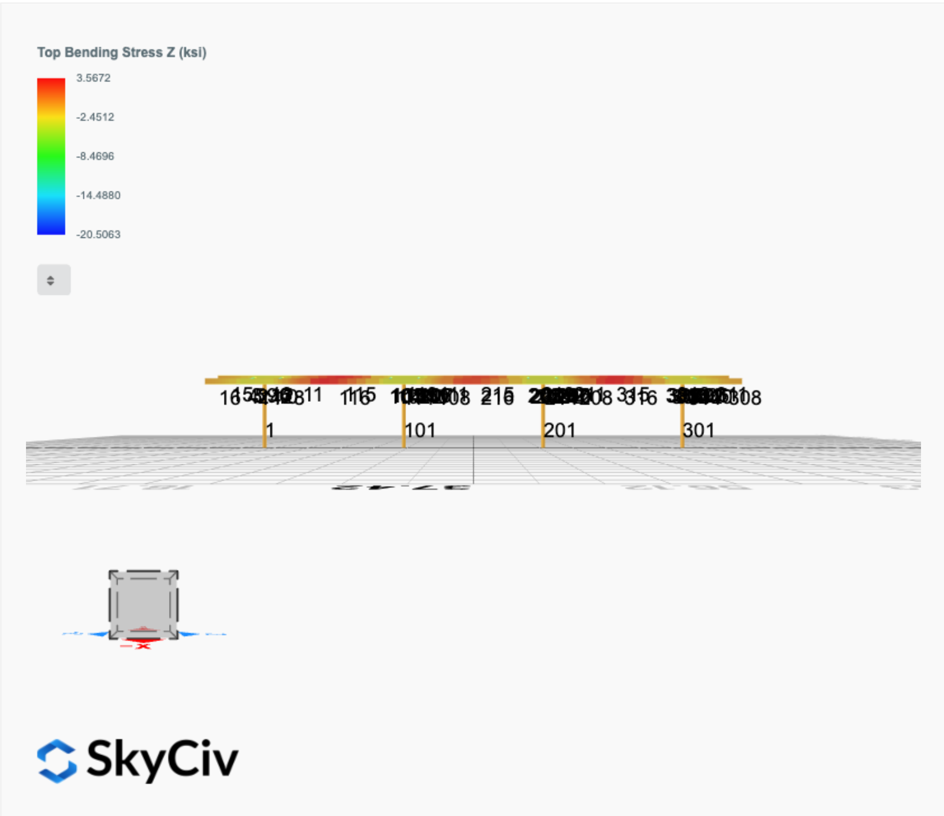


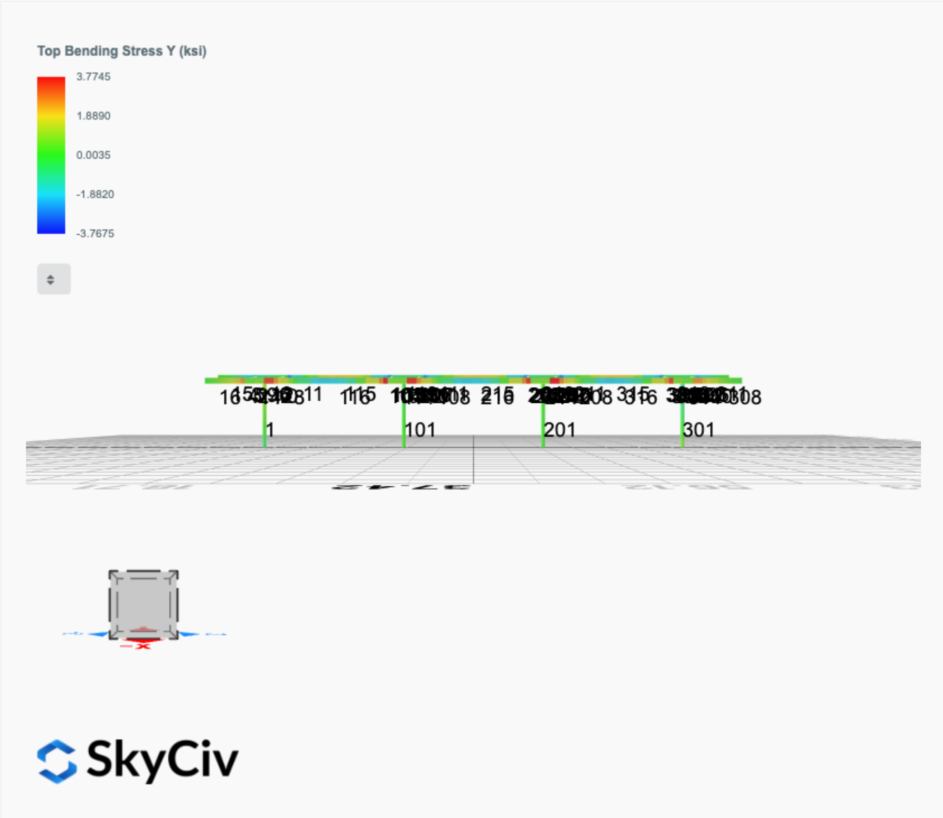
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	9.02884484	0.262	PASS
Material Yield	34.5	9.02884484	0.262	PASS
Material Strength	37	9.02884484	0.244	PASS

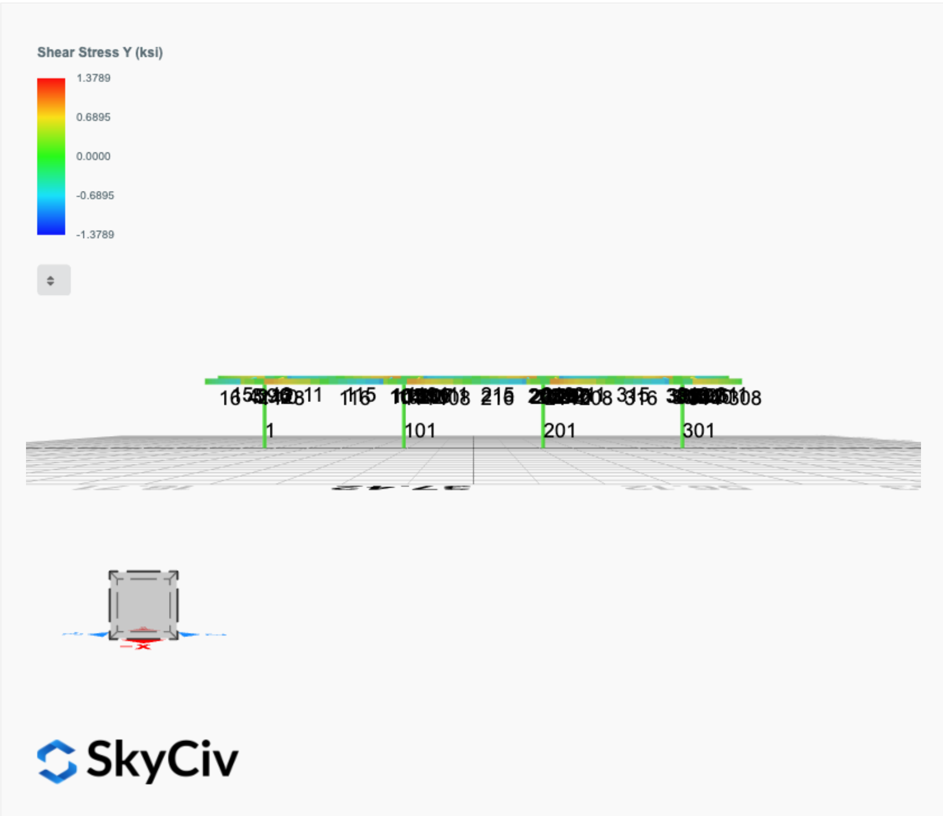


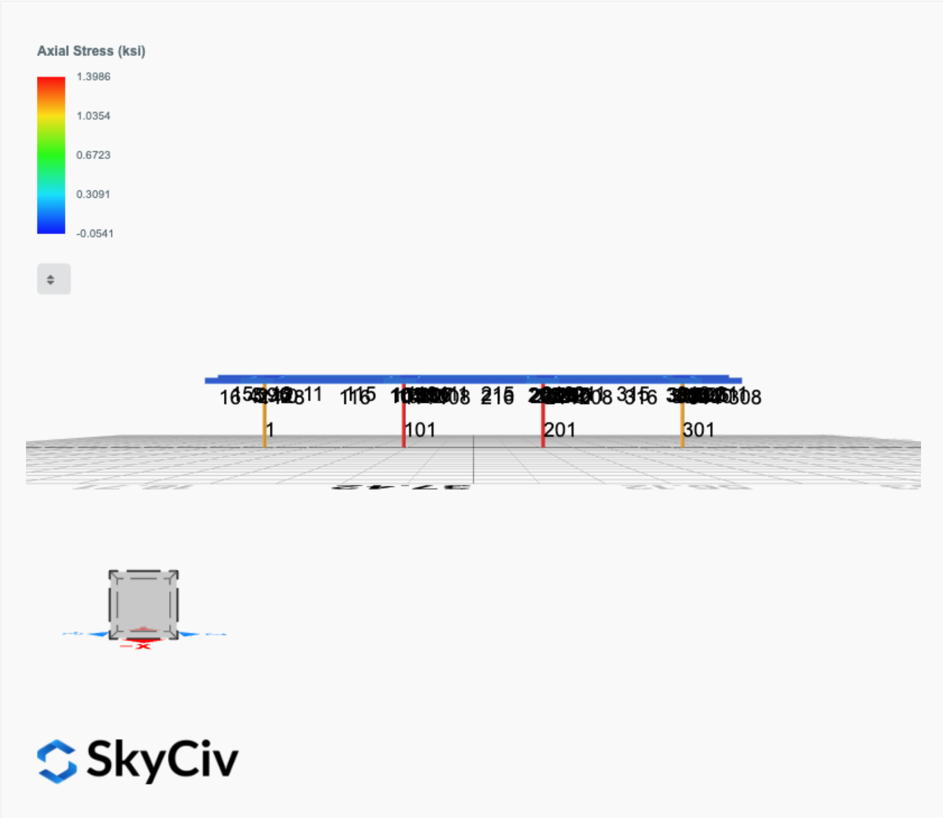
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
3.7745
1.8890
0.0035
-1.8820
-3.7675
1
101
201
301
SkyCiv


Shear Stress Y (ksi)
1.3789
0.6895
0.0000
-0.6895
-1.3789
1
101
201
301
SkyCiv


Axial Stress (ksi)
1.3986
1.0354
0.6723
0.3091
-0.0541
1
101
201
301

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	0.0017	2.0572	0.0215	0.0643	-0.0008	0.0186
ULS: 2. D + L	0.0017	2.0572	0.0215	0.0643	-0.0008	0.0186
ULS: 3. D + (S or Lr or R)	0.0086	8.7423	0.1122	0.3360	-0.0042	-0.0324
ULS: 3. D + (S or Lr or R)	0.0017	2.0572	0.0215	0.0643	-0.0008	0.0186
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0069	7.0710	0.0895	0.2681	-0.0034	-0.0196
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0017	2.0572	0.0215	0.0643	-0.0008	0.0186
ULS: 5b. D + 0.7E	0.0017	2.0572	0.0215	0.0643	-0.0008	0.0186
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0069	7.0710	0.0895	0.2681	-0.0034	-0.0196
ULS: 8. 0.6D + 0.7E	0.0010	1.2343	0.0129	0.0386	-0.0005	0.0111
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1759	4.0506	0.0492	0.1465	-0.0083	2.1595
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1759	4.0506	0.0492	0.1465	-0.0083	2.1595
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0457	1.5200	0.0153	0.0457	-0.0005	1.6540
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1219	0.7900	0.0017	0.0057	0.0074	-5.7039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1263	8.5661	0.1103	0.3297	-0.0090	1.5861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1263	8.5661	0.1103	0.3297	-0.0090	1.5861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0399	6.6682	0.0848	0.2541	-0.0031	1.2070
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0970	6.1207	0.0746	0.2241	0.0028	-4.3115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1315	3.5522	0.0423	0.1259	-0.0064	1.6243
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1315	3.5522	0.0423	0.1259	-0.0064	1.6243
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0347	1.6543	0.0168	0.0503	-0.0006	1.2452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0918	1.1068	0.0066	0.0204	0.0053	-4.2733
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1766	3.2277	0.0406	0.1207	-0.0080	2.1521
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1766	3.2277	0.0406	0.1207	-0.0080	2.1521
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0451	0.6972	0.0067	0.0200	-0.0002	1.6466
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1212	-0.0328	-0.0070	-0.0200	0.0077	-5.7113

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.8259
Shear X	-0.2960
Shear Z	0.1947
Moment X	0.5853
Moment Y (Twist)	0.0154
Moment Z	9.9988

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	8.7423
Shear X	-0.1766
Shear Z	0.1122
Moment X	0.3360
Moment Y (Twist)	0.0090
Moment Z	5.7113

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	-0.0017	2.3254	-0.0022	-0.0065	0.0004	0.0468
ULS: 2. D + L	-0.0017	2.3254	-0.0022	-0.0065	0.0004	0.0468
ULS: 3. D + (S or Lr or R)	-0.0086	10.1316	-0.0113	-0.0344	0.0020	0.1179
ULS: 3. D + (S or Lr or R)	-0.0017	2.3254	-0.0022	-0.0065	0.0004	0.0468
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0069	8.1801	-0.0090	-0.0274	0.0016	0.1001
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0017	2.3254	-0.0022	-0.0065	0.0004	0.0468
ULS: 5b. D + 0.7E	-0.0017	2.3254	-0.0022	-0.0065	0.0004	0.0468

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0069	8.1801	-0.0090	-0.0274	0.0016	0.1001
ULS: 8. 0.6D + 0.7E	-0.0010	1.3953	-0.0013	-0.0039	0.0002	0.0281
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2021	4.6526	-0.0045	-0.0137	-0.0003	2.4281
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2021	4.6526	-0.0045	-0.0137	-0.0003	2.4281
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0563	1.6967	-0.0009	-0.0028	-0.0010	1.7853
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1181	0.8493	-0.0019	-0.0053	0.0037	-6.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1572	9.9254	-0.0108	-0.0328	0.0011	1.8862
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1572	9.9254	-0.0108	-0.0328	0.0011	1.8862
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0366	7.7085	-0.0081	-0.0246	0.0006	1.4040
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0830	7.0730	-0.0088	-0.0265	0.0041	-4.5455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1520	4.0708	-0.0039	-0.0119	-0.0001	1.8328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1520	4.0708	-0.0039	-0.0119	-0.0001	1.8328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0418	1.8539	-0.0012	-0.0037	-0.0006	1.3506
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0882	1.2184	-0.0019	-0.0056	0.0029	-4.5989
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2014	3.7224	-0.0036	-0.0111	-0.0004	2.4094
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2014	3.7224	-0.0036	-0.0111	-0.0004	2.4094
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0569	0.7665	-0.0000	-0.0002	-0.0011	1.7666
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1188	-0.0808	-0.0010	-0.0027	0.0036	-6.1662

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	17.2198
Shear X	-0.3391
Shear Z	-0.0192
Moment X	-0.0587
Moment Y (Twist)	0.0073
Moment Z	10.7634

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	10.1316
Shear X	-0.2021
Shear Z	-0.0113
Moment X	-0.0344
Moment Y (Twist)	0.0041
Moment Z	6.1662

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	-0.0017	2.3254	0.0022	0.0065	-0.0004	0.0468
ULS: 2. D + L	-0.0017	2.3254	0.0022	0.0065	-0.0004	0.0468
ULS: 3. D + (S or Lr or R)	-0.0086	10.1316	0.0113	0.0344	-0.0020	0.1179
ULS: 3. D + (S or Lr or R)	-0.0017	2.3254	0.0022	0.0065	-0.0004	0.0468
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0069	8.1801	0.0090	0.0274	-0.0016	0.1001
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0017	2.3254	0.0022	0.0065	-0.0004	0.0468
ULS: 5b. D + 0.7E	-0.0017	2.3254	0.0022	0.0065	-0.0004	0.0468
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0069	8.1801	0.0090	0.0274	-0.0016	0.1001
ULS: 8. 0.6D + 0.7E	-0.0010	1.3953	0.0013	0.0039	-0.0002	0.0281
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2021	4.6526	0.0045	0.0137	0.0003	2.4281
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2021	4.6526	0.0045	0.0137	0.0003	2.4281
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0563	1.6967	0.0009	0.0028	0.0010	1.7853
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1181	0.8493	0.0019	0.0053	-0.0037	-6.1475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1572	9.9254	0.0108	0.0328	-0.0011	1.8862
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1572	9.9254	0.0108	0.0328	-0.0011	1.8862
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0366	7.7085	0.0081	0.0246	-0.0006	1.4040
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0830	7.0730	0.0088	0.0265	-0.0041	-4.5455

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1520	4.0708	0.0039	0.0119	0.0001	1.8328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1520	4.0708	0.0039	0.0119	0.0001	1.8328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0418	1.8539	0.0012	0.0037	0.0006	1.3506
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0882	1.2184	0.0019	0.0056	-0.0029	-4.5989
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2014	3.7224	0.0036	0.0111	0.0004	2.4094
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2014	3.7224	0.0036	0.0111	0.0004	2.4094
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0569	0.7665	0.0000	0.0002	0.0011	1.7666
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1188	-0.0808	0.0010	0.0027	-0.0036	-6.1662

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	17.2198
Shear X	-0.3391
Shear Z	0.0192
Moment X	0.0588
Moment Y (Twist)	0.0073
Moment Z	10.7634

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	10.1316
Shear X	-0.2021
Shear Z	0.0113
Moment X	0.0344
Moment Y (Twist)	0.0041
Moment Z	6.1662

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	0.0017	2.0572	-0.0215	-0.0643	0.0008	0.0186
ULS: 2. D + L	0.0017	2.0572	-0.0215	-0.0643	0.0008	0.0186
ULS: 3. D + (S or Lr or R)	0.0086	8.7423	-0.1122	-0.3360	0.0042	-0.0324
ULS: 3. D + (S or Lr or R)	0.0017	2.0572	-0.0215	-0.0643	0.0008	0.0186
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0069	7.0710	-0.0895	-0.2681	0.0034	-0.0196
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0017	2.0572	-0.0215	-0.0643	0.0008	0.0186
ULS: 5b. D + 0.7E	0.0017	2.0572	-0.0215	-0.0643	0.0008	0.0186
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0069	7.0710	-0.0895	-0.2681	0.0034	-0.0196
ULS: 8. 0.6D + 0.7E	0.0010	1.2343	-0.0129	-0.0386	0.0005	0.0111
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.1759	4.0506	-0.0492	-0.1465	0.0083	2.1595
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.1759	4.0506	-0.0492	-0.1465	0.0083	2.1595
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0457	1.5200	-0.0153	-0.0457	0.0005	1.6540
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1219	0.7900	-0.0017	-0.0057	-0.0074	-5.7039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1263	8.5661	-0.1103	-0.3297	0.0090	1.5861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1263	8.5661	-0.1103	-0.3297	0.0090	1.5861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0399	6.6682	-0.0848	-0.2541	0.0032	1.2070
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0970	6.1207	-0.0746	-0.2241	-0.0028	-4.3114
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1315	3.5522	-0.0423	-0.1259	0.0064	1.6243
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1315	3.5522	-0.0423	-0.1259	0.0064	1.6243
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0347	1.6543	-0.0168	-0.0503	0.0006	1.2452
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0918	1.1068	-0.0066	-0.0204	-0.0053	-4.2733
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.1766	3.2277	-0.0406	-0.1207	0.0080	2.1521
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.1766	3.2277	-0.0406	-0.1207	0.0080	2.1521
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0451	0.6972	-0.0067	-0.0200	0.0002	1.6466
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1212	-0.0328	0.0070	0.0200	-0.0077	-5.7113

Worst Case Reactions LRFD

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	14.8259
Shear X	-0.2960
Shear Z	-0.1947
Moment X	-0.5853
Moment Y (Twist)	0.0154
Moment Z	9.9989

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	8.7423
Shear X	-0.1766
Shear Z	-0.1122
Moment X	-0.3360
Moment Y (Twist)	0.0090
Moment Z	5.7113

REFERENCES	CALCULATIONS	RESULTS

Wind Load Calculations based on ASCE 7-16

Design Information :

Project Name : W11232. Revised2
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **Spearfish, South Dakota, 57783 USA** categorized as **Exposure B** (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Morever, the structure is classified as **Risk Category I**. The location is elevated at **4196.02 ft** above mean sea level.

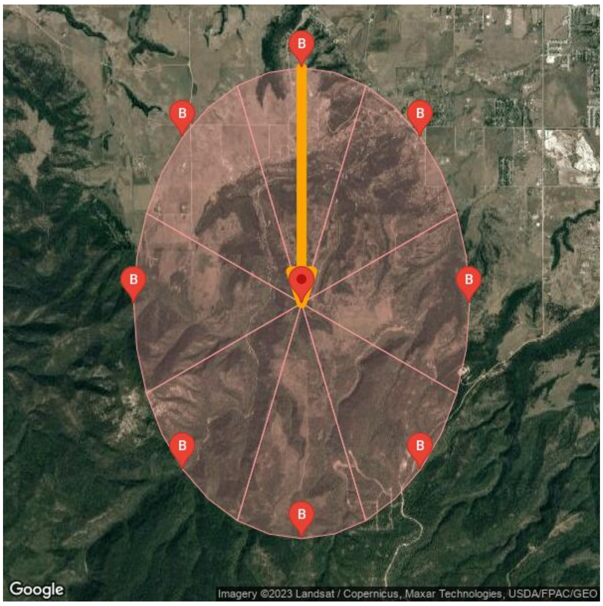


Figure 1. Site location.

Parameter	Value
Building Length, L	14.84 ft
Building Width, B	74.83 ft
Mean Roof Height, h	9.66 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	5.00°
Structure Type	Main Wind Force Resisting System (MWFRS)
Wind Blockage	Empty Under

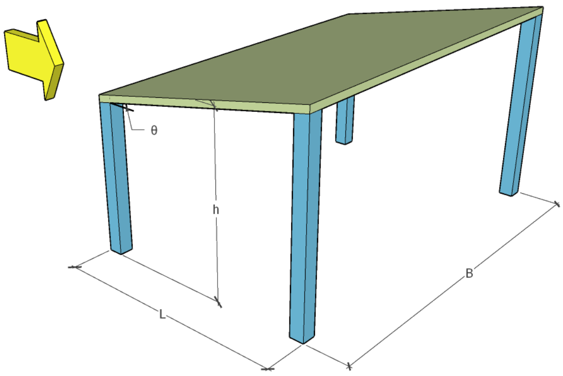


Figure 2. Building parameters.

Figure 26.5-1	**Basic Wind Speed, V** Wind speed for the address is **115 mph (defined by the user)** for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.	$V = 115$ mph **(defined by the user)**
Table 207A.6-1	**Wind Directionality Factor, K_d** $K_d = 0.85$ - Wind Directionality Factor *For buildings*	$K_d = 0.85$
Figure 26.8-1	**Topographic Factor, K_{zt}** $H/(L_h) = 0.08089$ *Slope of the hill, ridge, or escarpment*	$H/(L_h) = 0.08089$

Figure 26.8-1 $L_h = 12012$ ft — $L_h = 12012$ ft

Distance upwind of crest to where the difference in ground elevation is half the length of hill or escarpment.

Figure 26.8-1 $H = 971.69$ ft — $H = 971.69$ ft

Height of the hill or escarpment relative to the upwind terrain

Figure 26.8-1 $K_{zt} = 1$ - Topographic Factor — $K_{zt} = 1$

For the selected wind source direction, the terrain is relatively a flat surface since $H/L_h = 0.081$ is less than 0.2.

For calculating the hill-shape multiplier, the detected topography for the selected wind source direction is Hill.

Ground Elevation Factor, K_e

Section 26.9 K_e - Ground Elevation Factor

$$K_e = e^{-0.0000362\,E}$$

$$K_e = 0.85908$$

$$\text{Where } E = \text{Site Elevation} = 4196 \text{ ft}$$

Section 26.9 $K_e = 0.85908$ - Ground Elevation Factor — $K_e = 0.85908$

Velocity Pressure Exposure Coefficient, K_z

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Level	Elevation (ft)	K_z
h	9.657	0.575

Velocity Pressure, q_h

For the selected wind source direction.

Section 26.10.2 q_h - Velocity Pressure at h

$$q_h = 0.00256\,K_{z,h}\,K_{zt}\,K_d\,K_e\,V^2$$

$$q_h = 14.208 \text{ psf}$$

$$\begin{aligned} \text{Where } K_{z,h} &= 0.57472 \\ K_{zt} &= \text{Topographic Factor} = 1 \\ K_d &= \text{Wind Directionality Factor} = 0.85 \\ V &= \text{Basic Wind Speed} = 115 \text{ mi/h} \\ K_e &= \text{Ground Elevation Factor} = 0.85908 \end{aligned}$$

Velocity Pressure for All Directions

Section 26.8 K_{zt} - Topographic Factor

$$K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $15ft \le z \le z_g$

$$K_z = 2.01 \times (z/z_g)^{2/\alpha}$$

Section 26.10 K_z - Velocity Pressure Exposure Coefficient

For $z < 15ft$

$$K_z = 2.01 \times (15/z_g)^{2/\alpha}$$

Direction	Exposure Category	K_z @ $h = 9.657ft$	K_{zt}	K_d	K_d	V (mph)	q_h (psf)
N	B	0.575	1.000	0.850	0.859	115.000	14.208
S	B	0.575	1.000	0.850	0.859	115.000	14.208
E	B	0.575	1.000	0.850	0.859	115.000	14.208
W	B	0.575	1.000	0.850	0.859	115.000	14.208
NE	B	0.575	1.000	0.850	0.859	115.000	14.208
SE	B	0.575	1.000	0.850	0.859	115.000	14.208
NW	B	0.575	1.000	0.850	0.859	115.000	14.208
SW	B	0.575	1.000	0.850	0.859	115.000	14.208

Net Pressure Coefficients, C_N

Figure 27.3-4 to 27.3-7 The net pressure coefficients, C_N, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.

Direction	Surface	C_N Case A	C_N Case B
0	Windward	-	-1.300
	Leeward	-0.567	-0.033
180	Windward	1.000	0.700
	Leeward	1.100	0.167
90	≤ h from windward edge	-0.800	0.800
	h to 2h from windward edge	-0.600	0.500
	> 2h from windward edge	-0.300	0.300

Gust Effect Factor, G

Section 26.11 $G = 0.85$ - Gust Effect Factor — $G = 0.85$

The structure is assumed to be rigid.

Design Wind Pressures (MWFRS)

Section 27.3.2

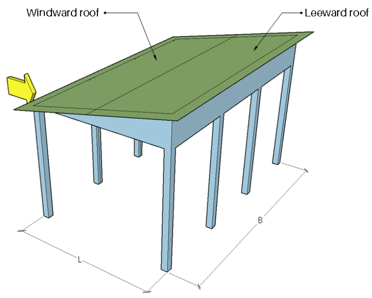
p - Design Wind Pressure

For open buildings

$$p = q_h \times G \times C_N$$

For Wind Pressure - 0°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	0.000	-15.700
	Leeward	-6.844	-0.403
180	Windward	12.077	8.454
	Leeward	13.285	2.013



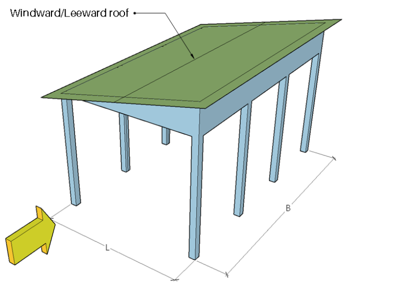
Wind along L - 0°

Service Wind Pressure - 0°/180°

Direction	Surface	p Case A (psf)	p Case B (psf)
0	Windward	0.000	-9.420
	Leeward	-4.106	-0.242
180	Windward	7.246	5.072
	Leeward	7.971	1.208

For Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-9.662	9.662
	h to 2h from windward edge	-7.246	6.039
	> 2h from windward edge	-3.623	3.623



Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	p Case A (psf)	p Case B (psf)
90	≤ h from windward edge	-5.797	5.797
	h to 2h from windward edge	-4.348	3.623
	> 2h from windward edge	-2.174	2.174

Section 27.3.2
Section 28.3.5

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ri dge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted and be reduced based on the actual solidity ratio ϕ and nu mber of frames n - See Figure 28.3-2.

Section 27.3.2
Section 28.3.5

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

	$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$	
Section 27.3.2 *Section 28.3.5*	K_B - Frame Width Factor *For $L < 100ft$, $K_B = 1.8 - 0.01L$. Otherwise, $K_B = 0.8$.* $$K_B = 1.8 - 0.01 * L \leq 0.8$$ $$K_B = 1.6516$$ $$\text{Where } L = \text{Building Length} = 14.843 \text{ ft}$$	
Section 28.3.5	K_S - Shielding Factor $$K_S = 0.6 + 0.073 \times (n-1) + \left(1.25 \times \phi^{1.8}\right)$$	
Section 28.3.5	$K_S = 1$ - Shielding Factor *Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n.*	$K_S = 1$
Figure 28.3-1 *Section 28.3.5*	$(GC_{pf})_{windward} = 0.4$ *Using Zone 5 from Figure 28.3-1*	$(GC_{pf})_{windward} = 0.4$
Figure 28.3-1 *Section 28.3.5*	$(GC_{pf})_{leeward} = -0.29$ *Using Zone 6 from Figure 28.3-1*	$(GC_{pf})_{leeward} = -0.29$
Section 27.3.2 *Section 28.3.5*	p - Horizontal Wind Loads on Open or Partially Enclosed Buildings *For wind pressure parallel to the ridge (90°)* $$p = q_h \ [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \ K_B \ K_S$$ $$p = 16.192 \text{ psf}$$	

REFERENCES	CALCULATIONS	RESULTS

Snow Load Detailed Calculations based on ASCE 7-16

Design Information :

Project Name : W11232. Revised2
Client :
Designer : MT_SKYCIV AutoDesigner
Company : MT Solar
Units : Imperial
Notes : Snow loads based on monoslope structure

Project Data

The structure is located in **Spearfish, South Dakota, 57783 USA** categorized as **Risk Category I**. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at **4196 ft** above mean sea level.

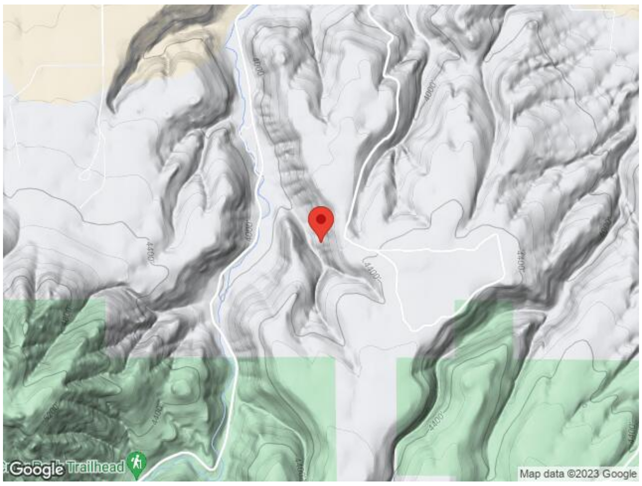


Figure 1. Site location.

Additional details of the structure are shown in Table below and illustrated in Figure 2:

Parameter	Value
Building Length, L	14.843 ft
Building Width, B	74.833 ft
Mean Roof Height, h	9.657 ft
Roof Profile	Open Monoslope
Roof Pitch Angle, θ	5.000°

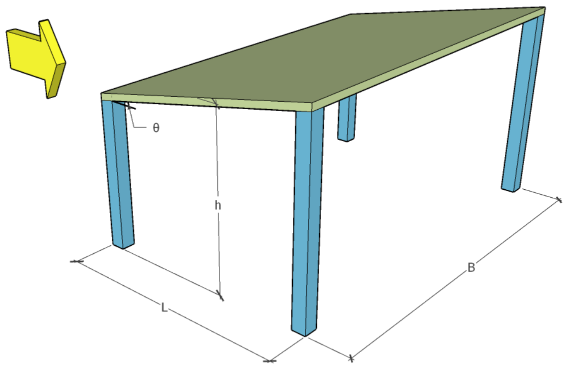


Figure 2. Building parameters.

REFERENCES	CALCULATIONS	RESULTS
Section 7.2 of ASCE 7	**Ground Snow Load, p_g** The ground snow load, p_g, for the site location is **43 psf (defined by the user)** at elevation 4196.02 ft above mean sea level based on Section 7.2 of ASCE 7	$p_g = 43$ psf **(defined by the user)**

	l based on Section 7.2 of ASCE 7.	**y the user)**
Table 7-2 Section 7.3.1 of ASCE 7	**Exposure Factor, C_e** The exposure factor, C_e, for the structure is equal 0.90 as the terrain is categorized as **Exposure B** with exposure condition specified as **Fully Exposed** based on Table 7-2 Section 7.3.1 of ASCE 7.	$C_e = 0.90$
Table 7-3 Section 7.3.2 of ASCE 7	**Thermal Factor, C_e** Since the thermal condition of the structure is categorzied as **"Unheated and open air structures,"** the corresponding thermal factor, C_t, is equal 1.20 based on Table 7-3 Section 7.3.2 of ASCE 7.	$C_t = 1.20$
Table 1.5-2 of Chapter 1 ASCE 7	**Importance Factor, I_s** Since the structure is classified Risk Category I, the Importance Factor, I_s, is equal to **0.8**.	$I_s = 0.80$
Equation 7.3-1 of Section 7.3 ASCE 7	**Flat Roof Snow Load, p_f** The flat roof snow load, p_f, (psf) is calculated using the Equation 7.3-1: $$p_f = 0.7C_eC_tI_sp_g$$ $$p_f = 0.7(0.90)(1.20)(0.80)(43.00) = 26.01psf$$	$p_f = 26.01$ psf
Section 7.10 ASCE 7	**Rain-on-snow Surcharge Load, p_r** The rain-on-snow surcharge load, p_r, is equal to 0.00 psf since p_g > 20 psf.	$p_r = 0.00$ psf
Equation 7.7-1 of ASCE 7	**Snow Density, γ** The snow density, γ, is calculated using Equation 7.7-1 of ASCE 7 as: $$\gamma = 0.13p_g + 14 \leq 30 = 0.13(43.00) + 14 \leq 30$$ $$\gamma = 19.59pcf$$	$\gamma = 19.59pcf$
Section 7.4 ASCE 7	**Roof Slope Factor (Balanced), C_s** Since the roof is classified as cold roof ($C_t > 1.0$), the corresponding roof slope factor, C_s, is equal to 1.000 based on Figure 7.2c where $\theta = 5.00°$.	$C_s = 1.000$
Equation 7.4-1 of Section 7.4 ASCE 7	**Sloped Roof Snow Load (Balanced), p_s** The sloped roof snow load, p_s, (psf) is calculated using the Equation 7.4-1: $$p_s = C_sp_f$$ $$p_s = (1.000)(26.01) = 26.01psf$$	$p_s = 26.01$ psf

Project Details

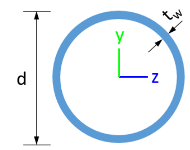
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

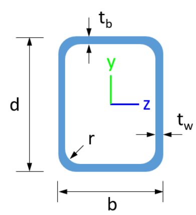
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

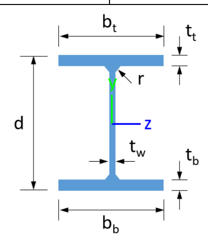
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
7	6in Pipe Sch 40	6.63	0.28					



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



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	62.42	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	20.28	20.28	9.66	-	300	200	1
2	6	4.20	4.20	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.16,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.19,1.17	300	200	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.68,1.68,1.94,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.55,1.69	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.36,1.67,1.67,1.68,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.19,1.19,1.19,1.01,1.18,1.18,1.19,1.17	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.49,1.67,1.67,1.68,1.66	300	200	1
8	20	1.33	1.33	2.05	1.62,1.63,1.62,1.63,1.63,1.62,1.62,1.62,1.57,1.75,1.61,1.61,1.06,2.06,1.62,1.62,1.62,1.65,1.61,1.61,1.53,1.80,1.61,1.61,2.17,2.08	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.68,1.68,1.97,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.56,1.69	300	200	1
11	20	1.33	1.33	2.05	1.64,1.64,1.64,1.64,1.64,1.64,1.65,1.65,1.69,1.31,1.66,1.66,1.78,2.03,1.64,1.64,1.65,1.61,1.65,1.65,1.70,1.65,1.66,1.66,1.84,1.93	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.11,1.12,1.12,1.13,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.13,2.64,1.12,1.12,1.13,1.13	300	200	1
14	20	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.03,1.14,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.13,1.12,1.12,1.53,1.19	300	200	1
15	20	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	20	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	7	20.28	20.28	9.66	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.56,1.18,1.18,1.19,1.17	300	200	1
104	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.86,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.70,1.69,1.67,1.67,1.56,1.69	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.25,1.67,1.67,1.68,1.66	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.08,1.18,1.18,1.19,1.17	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.43,1.67,1.67,1.68,1.66	300	200	1

108	20	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.11,2.06,2.10,2.10,1.26,1.72,2.09,2.09,2.09,2.08,2.09,2.09,2.21,2.02,2.10,2.10,1.49,1.60	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.89,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.57,1.69	300	200	1
111	20	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.08,2.08,2.08,1.76,2.08,2.08,2.06,1.74,2.09,2.09,2.09,2.10,2.08,2.08,2.07,1.16,2.08,2.08,1.95,1.82	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1
113	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.03,1.05,1.05,1.05,1.14,1.05,1.05,1.05,1.04,1.05,1.05,1.05,1.03,1.05,1.05,1.05,1.09	300	200	1
114	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,1.02,1.14,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,2.41,1.28	300	200	1
115	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.15,1.28,1.15,1.15,1.13,1.11,1.15,1.15,1.15,1.16,1.15,1.15,1.14,1.16,1.15,1.15,1.13,1.12	300	200	1
116	20	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.16,1.16,1.16,1.14,1.16,1.16,1.76,1.11,1.15,1.15,1.15,1.15,1.16,1.16,1.17,1.13,1.16,1.16,1.10,1.10	300	200	1
201	7	20.28	20.28	9.66	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.08,1.18,1.18,1.19,1.17	300	200	1
204	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.89,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.57,1.69	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.43,1.67,1.67,1.68,1.66	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.56,1.18,1.18,1.19,1.17	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.25,1.67,1.67,1.68,1.66	300	200	1
208	20	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.09,1.90,2.08,2.08,1.43,1.58,2.07,2.07,2.07,2.07,2.07,2.07,2.10,1.81,2.08,2.08,1.37,1.48	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.86,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.70,1.69,1.67,1.67,1.56,1.69	300	200	1
211	20	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.05,2.09,2.06,2.06,1.84,1.60,2.07,2.07,2.07,2.08,2.06,2.06,1.99,1.05,2.06,2.06,1.76,1.66	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.03,1.05,1.05,1.05,1.14,1.05,1.05,1.05,1.04,1.05,1.05,1.05,1.03,1.05,1.05,1.05,1.09	300	200	1
214	20	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,1.02,1.14,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,2.41,1.28	300	200	1
215	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.57,1.15,1.15,1.14,1.13,1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.06,1.15,1.15,1.14,1.13	300	200	1
216	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.15,1.16,1.16,2.47,1.13,1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.14,1.16,1.16,1.11,1.12	300	200	1
301	7	20.28	20.28	9.66	-	300	200	1

302	6	1.30	1.30	2.00	-	300	200	1
303	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.19,1.19,1.19,1.01,1.18,1.18,1.19,1.17	300	200	1
304	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.68,1.68,1.97,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.56,1.69	300	200	1
305	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.49,1.67,1.67,1.68,1.66	300	200	1
306	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.20,1.18,1.18,1.19,1.16,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.18,1.18,1.19,1.17	300	200	1
307	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.36,1.67,1.67,1.68,1.66	300	200	1
308	20	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
309	3	2.60	2.60	4.00	-	300	200	1
310	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.68,1.68,1.94,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.71,1.69,1.67,1.67,1.55,1.69	300	200	1
311	20	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
312	6	4.20	4.20	2.00	-	300	200	1
313	20	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.11,1.12,1.12,1.13,1.13,1.12,1.12,1.12,1.12,1.12,1.12,1.13,2.64,1.12,1.12,1.13,1.13	300	200	1
314	20	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.12,1.12,1.03,1.14,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.13,1.12,1.12,1.53,1.19	300	200	1
315	20	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.13,1.15,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.53,1.12,1.12,1.13,1.14	300	200	1
316	20	6.63	6.63	10.20	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.13,1.11,1.11,1.22,1.15,1.11,1.11,1.11,1.12,1.11,1.11,1.10,1.13,1.11,1.11,1.18,1.16	300	200	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	106.42	42.30	42.30	75.35	75.35
2	251.01	229.64	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	33.92	6.46	56.26	44.91
14	159.30	116.35	31.48	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	251.16	106.42	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95

103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.48	6.46	56.26	44.91
114	159.30	116.35	31.17	6.46	56.26	44.91
115	159.30	75.13	21.38	6.46	56.26	44.91
116	159.30	75.13	21.18	6.46	56.26	44.91
201	251.16	106.42	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	142.47	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.48	6.46	56.26	44.91
214	159.30	116.35	31.17	6.46	56.26	44.91
215	159.30	75.13	20.41	6.46	56.26	44.91
216	159.30	75.13	21.38	6.46	56.26	44.91
301	251.16	106.42	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	55.15	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	55.15	46.90	6.46	56.26	44.91
312	251.01	229.64	27.16	27.16	75.30	75.30
313	159.30	116.35	33.92	6.46	56.26	44.91
314	159.30	116.35	31.48	6.46	56.26	44.91
315	159.30	75.13	21.38	6.46	56.26	44.91
316	159.30	75.13	21.18	6.46	56.26	44.91

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.139	0.236	0.030	0.004	0.003	0.256	#16	0.542	Not Required	Pass
2	0.001	0.444	0.015	0.093	0.002	0.460	#21	0.116	Not Required	Pass
3	0.002	0.649	0.006	0.065	0.001	0.652	#21	0.046	Not Required	Pass
4	0.001	0.640	0.030	0.064	0.007	0.666	#21	0.082	Not Required	Pass

4	0.001	0.640	0.030	0.064	0.007	0.666	#21	0.082	Not Required	Pass
5	0.002	0.402	0.024	0.065	0.006	0.406	#21	0.076	Not Required	Pass
6	0.002	0.695	0.022	0.070	0.005	0.718	#21	0.046	Not Required	Pass
7	0.002	0.431	0.042	0.069	0.011	0.444	#21	0.076	Not Required	Pass
8	0.000	0.083	0.040	0.043	0.004	0.100	#21	0.102	Not Required	Pass
9	0.003	0.077	0.018	0.001	0.001	0.096	#21	0.206	Not Required	Pass
10	0.002	0.684	0.034	0.068	0.007	0.700	#21	0.082	Not Required	Pass
11	0.001	0.083	0.043	0.044	0.004	0.098	#21	0.102	Not Required	Pass
12	0.000	0.491	0.016	0.100	0.002	0.507	#21	0.054	Not Required	Pass
13	0.001	0.209	0.105	0.057	0.006	0.260	#21	0.306	Not Required	Pass
14	0.002	0.208	0.102	0.056	0.006	0.258	#21	0.204	Not Required	Pass
15	0.000	0.063	0.035	0.028	0.003	0.098	#21	Not Required	Not Required	Pass
16	0.000	0.062	0.035	0.028	0.003	0.097	#21	Not Required	Not Required	Pass
101	0.162	0.254	0.003	0.004	0.000	0.275	#16	0.542	Not Required	Pass
102	0.001	0.547	0.019	0.113	0.002	0.567	#21	0.054	Not Required	Pass
103	0.002	0.785	0.014	0.079	0.002	0.800	#21	0.046	Not Required	Pass
104	0.002	0.776	0.036	0.078	0.008	0.800	#21	0.082	Not Required	Pass
105	0.002	0.487	0.038	0.078	0.010	0.497	#21	0.076	Not Required	Pass
106	0.002	0.781	0.013	0.078	0.002	0.794	#21	0.046	Not Required	Pass
107	0.002	0.484	0.036	0.078	0.009	0.494	#21	0.076	Not Required	Pass
108	0.000	0.052	0.038	0.045	0.004	0.078	#21	0.102	Not Required	Pass
109	0.003	0.081	0.010	0.001	0.000	0.093	#21	0.206	Not Required	Pass
110	0.002	0.771	0.036	0.077	0.008	0.796	#21	0.082	Not Required	Pass
111	0.001	0.054	0.039	0.046	0.004	0.078	#21	0.102	Not Required	Pass
112	0.001	0.543	0.019	0.113	0.002	0.562	#21	0.036	Not Required	Pass
113	0.001	0.244	0.103	0.060	0.006	0.317	#21	0.306	Not Required	Pass
114	0.002	0.248	0.103	0.060	0.006	0.317	#21	0.306	Not Required	Pass
115	0.002	0.314	0.054	0.047	0.004	0.370	#21	0.507	Not Required	Pass
116	0.001	0.311	0.056	0.047	0.004	0.367	#21	0.507	Not Required	Pass
201	0.162	0.254	0.003	0.004	0.000	0.275	#16	0.542	Not Required	Pass
202	0.001	0.543	0.019	0.113	0.002	0.562	#21	0.036	Not Required	Pass
203	0.002	0.781	0.013	0.078	0.002	0.794	#21	0.046	Not Required	Pass
204	0.002	0.771	0.036	0.077	0.008	0.796	#21	0.082	Not Required	Pass
205	0.002	0.484	0.036	0.078	0.009	0.494	#21	0.076	Not Required	Pass
206	0.002	0.785	0.014	0.079	0.002	0.800	#21	0.046	Not Required	Pass
207	0.002	0.487	0.038	0.078	0.010	0.497	#21	0.076	Not Required	Pass
208	0.000	0.064	0.041	0.047	0.004	0.076	#21	0.102	Not Required	Pass
209	0.003	0.081	0.010	0.001	0.000	0.093	#21	0.206	Not Required	Pass
210	0.002	0.776	0.036	0.078	0.008	0.800	#21	0.082	Not Required	Pass
211	0.001	0.066	0.041	0.047	0.004	0.078	#21	0.102	Not Required	Pass
212	0.001	0.547	0.019	0.113	0.002	0.567	#21	0.054	Not Required	Pass
213	0.001	0.244	0.103	0.060	0.006	0.317	#21	0.306	Not Required	Pass
214	0.002	0.248	0.103	0.060	0.006	0.317	#21	0.306	Not Required	Pass
215	0.002	0.269	0.055	0.046	0.004	0.324	#21	0.507	Not Required	Pass
216	0.001	0.263	0.055	0.045	0.004	0.318	#21	0.507	Not Required	Pass
301	0.139	0.236	0.030	0.004	0.003	0.256	#16	0.542	Not Required	Pass
302	0.000	0.491	0.016	0.100	0.002	0.507	#21	0.054	Not Required	Pass
303	0.002	0.695	0.022	0.070	0.005	0.718	#21	0.046	Not Required	Pass
304	0.002	0.684	0.034	0.068	0.007	0.700	#21	0.082	Not Required	Pass
305	0.002	0.431	0.042	0.069	0.011	0.444	#21	0.076	Not Required	Pass
306	0.002	0.649	0.006	0.065	0.001	0.652	#21	0.046	Not Required	Pass
307	0.002	0.402	0.024	0.065	0.006	0.406	#21	0.076	Not Required	Pass
308	0.000	0.062	0.035	0.028	0.003	0.097	#21	Not Required	Not Required	Pass
309	0.003	0.077	0.018	0.001	0.001	0.096	#21	0.206	Not Required	Pass

310	0.001	0.640	0.030	0.064	0.007	0.666	#21	0.082	Not Required	Pass
311	0.000	0.063	0.035	0.028	0.003	0.098	#21	Not Required	Not Required	Pass
312	0.001	0.444	0.015	0.093	0.002	0.460	#21	0.116	Not Required	Pass
313	0.001	0.209	0.105	0.057	0.006	0.260	#21	0.204	Not Required	Pass
314	0.002	0.208	0.102	0.056	0.006	0.258	#21	0.306	Not Required	Pass
315	0.002	0.321	0.054	0.044	0.004	0.376	#21	0.507	Not Required	Pass
316	0.001	0.320	0.055	0.043	0.004	0.375	#21	0.507	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
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

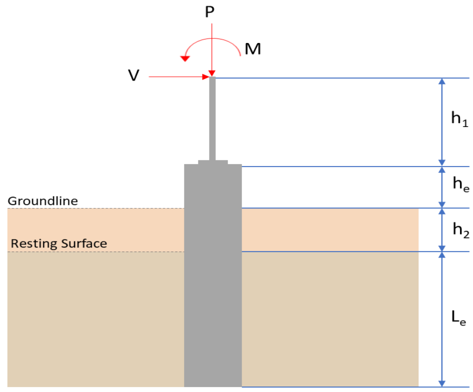
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - 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)	8.742	14.826
V_x (kip)	-0.177	-0.296
V_z (kip)	0.112	0.195
M_x (kipft)	0.336	0.585
M_z (kipft)	5.711	9.999

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.177\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.059\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.711 \text{ kipft}) + ((-0.177 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.9037 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.9094 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.112 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.037333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.336 \text{ kipft}) + ((0.112 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.112 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 2.895 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.9094 \text{ ft}), (2.895 \text{ ft})]$$

$$L_{e,req} = 5.909 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.909 \text{ ft})}{(6.25 \text{ ft})}$$

$$Ratio = 0.94544$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(8.742 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2367\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.2367\ \mathrm{kip/ft^2}\right)}{\left(2000\ \mathrm{psf}\right)}$$

$$Ratio = 0.61837$$

Status: **PASS**
Ratio: **0.620**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(36\ \mathrm{in})}$$

$$L/D = 2.0833$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.059\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 1.9037\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9037\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (1.9037\ \mathrm{kipft/ft})) + (4 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2262\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.9037\ \mathrm{kipft/ft})) + (3 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (1.9037\ \mathrm{kipft/ft})) + (2 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.25685\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (1.9037\ \mathrm{kipft/ft})) + ((-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.82966\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2262\ \mathrm{ft})}{2}$$

$$p_a = 0.31697\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.25685\ \mathrm{kip/ft^2}\right)}{\left(0.31697\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.81032$$

Status: **PASS**
Ratio: **0.810**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.82966\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88497$$

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

Considering z-direction:

$H_o = 0.037333\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.112\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.112\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.112\ \mathrm{kipft/ft})) + (4 \times (0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4695\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times \left[(4 \times (0.112\ \mathrm{kipft/ft})) + (3 \times (0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))\right]^2}{(6.25\ \mathrm{ft})^2 \times \left[(3 \times (0.112\ \mathrm{kipft/ft})) + (2 \times (0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))\right]}$$

$$p = 0.049515\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times \left[(2 \times (0.112\ \mathrm{kipft/ft})) + ((0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))\right]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.11035\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4695\ \mathrm{ft})}{2}$$

$$p_a = 0.33521\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.049515\ \mathrm{kip/ft^2}\right)}{\left(0.33521\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.14771$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

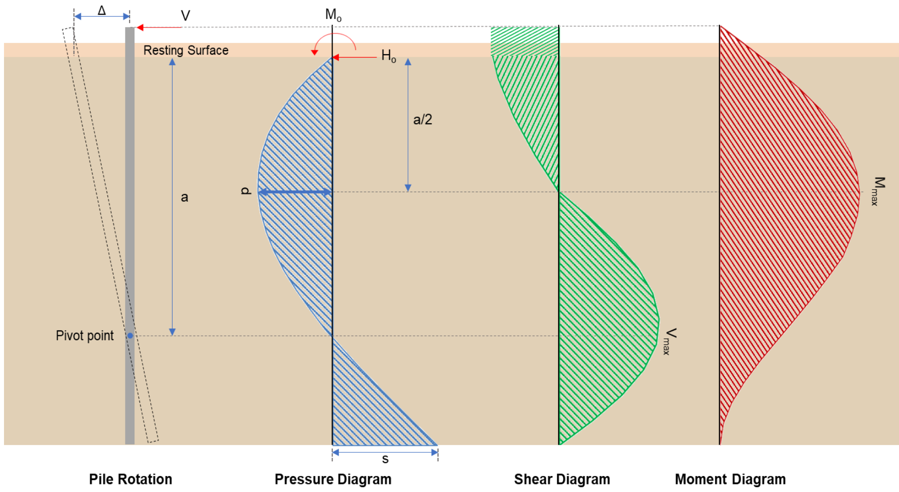
$$Ratio = \frac{s}{\ldots}$$

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

$$Ratio = \frac{(0.11035\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.1177$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.296\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.098667\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.999\ \mathrm{kipft}) + ((-0.296\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 3.333\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.333\ \mathrm{kipft/ft})}{(-0.098667\ \mathrm{kip/ft})}$$

$$E = 33.78\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.333\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.098667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (3.333\ \mathrm{kipft/ft})) + (4 \times (-0.098667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2239\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.098667\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (33.78\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.2239\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (33.78\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.2239\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 3.0323\ \text{kip}$$

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

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

$$M_{max} = ((-0.098667\ \text{kip/ft}) \times (36\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(33.78\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.2239\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (33.78\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.2239\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (33.78\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.2239\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 9.3062\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.195\ \text{kip})}{(36\ \text{in})}$$

$$H_o = 0.065\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.585\ \text{kipft}) + ((0.195\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 0.195\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.195\ \text{kipft/ft})}{(0.065\ \text{kip/ft})}$$

$$E = 3\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.195\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (0.065\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.195\ \text{kipft/ft})) + (4 \times (0.065\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.4695\ \text{ft}$$

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

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

$$V_{max} = ((0.065\ \text{kip/ft}) \times (36\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.4695\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.4695\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.29563\ \text{kip}$$

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

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

$$M_{max} = ((0.065\ \text{kip/ft}) \times (36\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(3\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.4695\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (3\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.4695\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.4695\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.81519 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(14.826 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (3 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -44.713 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-44.713 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\ \text{ksi}) \times \left[\left(1017.9\ \text{in}^2\right) - \left(1.8408\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(1.8408\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 1492.5\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(14.826\ \text{kip})}{(1492.5\ \text{kip})}$$

$$Ratio = 0.0099337$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\ \text{in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\ \text{in})$$

$$d = 28.8\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\ \text{psi})} \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,max} = 203.86\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$, $P = 14.826\ \text{kip} \rightarrow 14826\ \text{lbf}$,
22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\ \text{psi})} + \frac{(14826\ \text{lbf})}{6 \times (1017.9\ \text{in}^2)}\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,a} = 84.06\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,
22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\ \text{psi})} + (0.05 \times (3000\ \text{psi}))\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,b} = 237.06\ \text{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(203.86\ \text{kip}), (84.06\ \text{kip}), (237.06\ \text{kip})\right]$$

$$V_c = 84.06 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((84.06 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 79.449 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.0323 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(3.0323 \text{ kip})}{(79.449 \text{ kip})}$$

$$Ratio = 0.038167$$

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

Considering z-direction:

$V_{max} = 0.29563 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.29563 \text{ kip})}{(79.449 \text{ kip})}$$

$$Ratio = 0.003721$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

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

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

$$\phi M_{n,1} = 67.947 kipft$$

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

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

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

$$\phi M_{n,2} = 632.67 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$$

$$\phi M_n = 67.947 \text{ kipft}$$

Considering x-direction:

$M_{max} = 9.3062 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.3062 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.13696$$

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

Considering z-direction:

$M_{max} = 0.81519 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.81519 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.011998$$

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

REFERENCES	CALCULATIONS	RESULTS

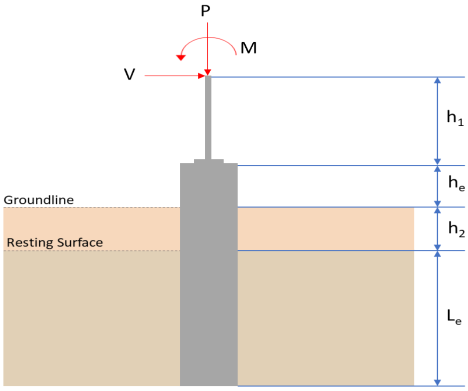
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - 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)	10.132	17.220
V_x (kip)	-0.202	-0.339
V_z (kip)	-0.011	-0.019
M_x (kipft)	-0.034	-0.059
M_z (kipft)	6.166	10.763

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.202\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.067333\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.166\ \mathrm{kipft}) + ((-0.202\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 2.0553\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.0366\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.011\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.0036667\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034\ \mathrm{kipft}) + ((-0.011\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 0.011333\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.0228\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.0366\ \mathrm{ft}), (1.0228\ \mathrm{ft})]$$

$$L_{e,req} = 6.037\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.25\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.037\ \mathrm{ft})}{(6.25\ \mathrm{ft})}$$

$$Ratio = 0.96592$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(10.132\ \mathrm{kip})}{(7.0686\ \mathrm{ft}^2)}$$

$$q = 1.4334\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.4334\ \mathrm{kip/ft^2}\right)}{\left(2000\ \mathrm{psf}\right)}$$

$$Ratio = 0.71669$$

Status: **PASS**
Ratio: **0.720**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(36\ \mathrm{in})}$$

$$L/D = 2.0833$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.067333\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.0553\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0553\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.067333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (2.0553\ \mathrm{kipft/ft})) + (4 \times (-0.067333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2292\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.0553\ \mathrm{kipft/ft})) + (3 \times (-0.067333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (2.0553\ \mathrm{kipft/ft})) + (2 \times (-0.067333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.27428\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (2.0553\ \mathrm{kipft/ft})) + ((-0.067333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.89028\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2292\ \mathrm{ft})}{2}$$

$$p_a = 0.31719\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.27428\ \mathrm{kip/ft^2}\right)}{\left(0.31719\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.86471$$

Status: **PASS**
Ratio: **0.860**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.89028\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.94964$$

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

Considering z-direction:

$H_o = -0.0036667\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.011333\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011333\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.011333\ \mathrm{kipft/ft})) + (4 \times (-0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4657\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011333\ \mathrm{kipft/ft})) + (3 \times (-0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.011333\ \mathrm{kipft/ft})) + (2 \times (-0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.0013974\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (0.011333\ \mathrm{kipft/ft})) + ((-0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = -0.00006032\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4657\ \mathrm{ft})}{2}$$

$$p_a = 0.33493\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0013974\ \mathrm{kip/ft^2}\right)}{\left(0.33493\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.0041723$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

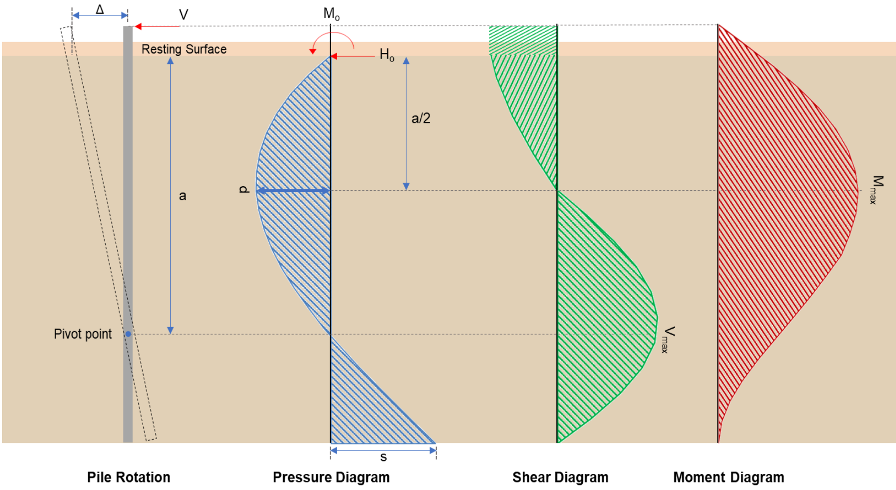
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.00006032\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.000064341$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.339\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.113\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.763\ \mathrm{kipft}) + ((-0.339\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 3.5877\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.5877\ \mathrm{kipft/ft})}{(-0.113\ \mathrm{kip/ft})}$$

$$E = 31.749\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.5877\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.113\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (3.5877\ \mathrm{kipft/ft})) + (4 \times (-0.113\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2271\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.113\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (31.749\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.2271\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (31.749\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.2271\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 3.2771 \text{ kip}$$

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

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

$$M_{max} = ((-0.113 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(31.749 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (31.749 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (31.749 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 10.046 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.019 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0063333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.059 \text{ kipft}) + ((-0.019 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.019667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.019667 \text{ kipft/ft})}{(-0.0063333 \text{ kip/ft})}$$

$$E = 3.1053 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.0063333 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.019667 \text{ kipft/ft})) + (4 \times (-0.0063333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4651 \text{ ft}$$

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

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

$$V_{max} = ((-0.0063333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4651 \text{ ft})}{(6.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4651 \text{ ft})}{(6.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.029364 \text{ kip}$$

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

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

$$M_{max} = ((-0.0063333 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.1053 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.081173 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(17.22 \text{ kip})}{(0.65)\times(0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2))\right]$$

$$A_{st,required} = -44.637 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-44.637 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$

$$\phi P_N = 1492.5\text{ kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(17.22\text{ kip})}{(1492.5\text{ kip})}$$

$$Ratio = 0.011538$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\text{ in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\text{ in})$$

$$d = 28.8\text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,max} = 203.86\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$, $P = 17.22\text{ kip} \to 17220\text{ lbf}$,
22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + \frac{(17220\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,a} = 84.466\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$,
22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,b} = 237.06\text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(203.86\text{ kip}), (84.466\text{ kip}), (237.06\text{ kip})\right]$$

$$V_c = 84.466 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((84.466 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 79.714 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.2771 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(3.2771 \text{ kip})}{(79.714 \text{ kip})}$$

$$Ratio = 0.041111$$

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

Considering z-direction:

$V_{max} = 0.029364 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.029364 \text{ kip})}{(79.714 \text{ kip})}$$

$$Ratio = 0.00036837$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$S_m = 4580.4\ \text{in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 4580.442in^3$$ $$\phi M_{n,1} = 67.947kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f_{ck}'\ S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3\ \text{ksi}) \times \left(4580.4\ \text{in}^3\right)$$ $$\phi M_{n,2} = 632.67\ \text{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(67.947\ \text{kipft}), (632.67\ \text{kipft})]$$ $$\phi M_n = 67.947\ \text{kipft}$$ **Considering x-direction:** $M_{max} = 10.046\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(10.046\ \text{kipft})}{(67.947\ \text{kipft})}$$ $$Ratio = 0.14786$$	Status: **PASS** Ratio: **0.150**
	Considering z-direction: $M_{max} = 0.081173\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.081173\ \text{kipft})}{(67.947\ \text{kipft})}$$ $$Ratio = 0.0011947$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

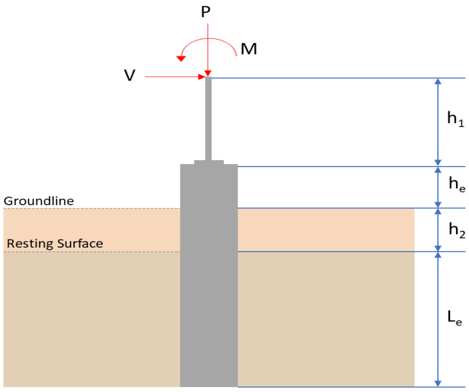
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - 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)	10.132	17.220
V_x (kip)	-0.202	-0.339
V_z (kip)	0.011	0.019
M_x (kipft)	0.034	0.059
M_z (kipft)	6.166	10.763

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.202\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.067333\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.166\ \text{kipft}) + ((-0.202\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 2.0553\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.0366\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.011\ \text{kip})}{(36\ \text{in})}$$

$$H_o = 0.0036667\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034\ \text{kipft}) + ((0.011\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 0.011333\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.2272\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.0366\ \text{ft}), (1.2272\ \text{ft})]$$

$$L_{e,req} = 6.037\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.037\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.96592$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(10.132\ \text{kip})}{(7.0686\ \text{ft}^2)}$$

$$q = 1.4334 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.4334 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.71669$$

Status: **PASS**
Ratio: **0.720**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.0833$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.067333 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 2.0553 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0553 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.067333 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (2.0553 \text{ kipft/ft})) + (4 \times (-0.067333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2292 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.0553 \text{ kipft/ft})) + (3 \times (-0.067333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^{\;2}}{(6.25 \text{ ft})^2 \times [(3 \times (2.0553 \text{ kipft/ft})) + (2 \times (-0.067333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.27428 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (2.0553 \text{ kipft/ft})) + ((-0.067333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89028 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2292 \text{ ft})}{2}$$

$$p_a = 0.31719 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.27428 \text{ kip/ft}^2)}{(0.31719 \text{ kip/ft}^2)}$$

$$Ratio = 0.86471$$

Status: **PASS**
Ratio: **0.860**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.89028\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.94964$$

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

Considering z-direction:

$H_o = 0.0036667\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.011333\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011333\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.011333\ \mathrm{kipft/ft})) + (4 \times (0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4657\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011333\ \mathrm{kipft/ft})) + (3 \times (0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^2}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.011333\ \mathrm{kipft/ft})) + (2 \times (0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.0049164\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (0.011333\ \mathrm{kipft/ft})) + ((0.0036667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.010998\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4657\ \mathrm{ft})}{2}$$

$$p_a = 0.33493\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0049164\ \mathrm{kip/ft^2}\right)}{\left(0.33493\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.014679$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

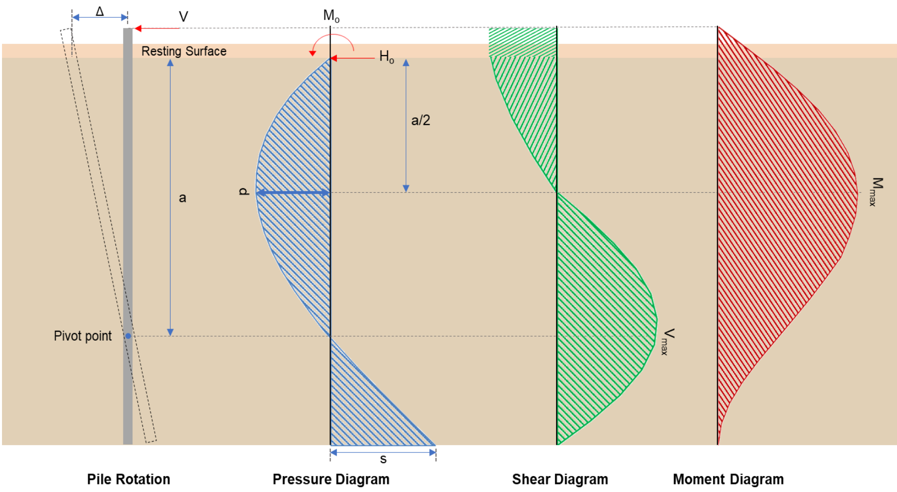
$$Ratio = \frac{s}{\ldots}$$

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

$$Ratio = \frac{(0.010998\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.011732$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.339\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.113\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(10.763\ \mathrm{kipft}) + ((-0.339\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 3.5877\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.5877\ \mathrm{kipft/ft})}{(-0.113\ \mathrm{kip/ft})}$$

$$E = 31.749\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.5877\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.113\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (3.5877\ \mathrm{kipft/ft})) + (4 \times (-0.113\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2271\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.113\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (31.749\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.2271\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (31.749\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.2271\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 3.2771 \text{ kip}$$

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

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

$$M_{max} = ((-0.113 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(31.749 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (31.749 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (31.749 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.2271 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 10.046 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.019 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0063333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.059 \text{ kipft}) + ((0.019 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.019667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.019667 \text{ kipft/ft})}{(0.0063333 \text{ kip/ft})}$$

$$E = 3.1053 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019667 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (0.0063333 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.019667 \text{ kipft/ft})) + (4 \times (0.0063333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4651 \text{ ft}$$

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

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

$$V_{max} = ((0.0063333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4651 \text{ ft})}{(6.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4651 \text{ ft})}{(6.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.029364 \text{ kip}$$

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

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

$$M_{max} = ((0.0063333 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.1053 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.1053 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4651 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.081173 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(17.22 \text{ kip})}{(0.65)\times(0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2))\right]$$

$$A_{st,required} = -44.637 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-44.637 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10⌀: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

	Axial Compression Strength (ACI 318-19, LRFD)	
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.85\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 1492.5\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(17.22\text{ kip})}{(1492.5\text{ kip})}$$ $$Ratio = 0.011538$$	Status: **PASS** Ratio: **0.010**
	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 36\text{ in}$ - Effective width,	
22.5.2.2	d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (36\text{ in})$$ $$d = 28.8\text{ in}$$	
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.71796$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,max} = 203.86\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$, $P = 17.22\text{ kip} \to 17220\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + \frac{(17220\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,a} = 84.466\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \to 3000\text{ psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,b} = 237.06\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(203.86\text{ kip}), (84.466\text{ kip}), (237.06\text{ kip})\right]$$	

$$V_c = 84.466 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((84.466 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 79.714 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.2771 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(3.2771 \text{ kip})}{(79.714 \text{ kip})}$$

$$Ratio = 0.041111$$

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

Considering z-direction:

$V_{max} = 0.029364 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.029364 \text{ kip})}{(79.714 \text{ kip})}$$

$$Ratio = 0.00036837$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

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

$$\phi M_{n,1} = 67.947kipft$$

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

$$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$

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

$$\phi M_{n,2} = 632.67 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$$

$$\phi M_n = 67.947 \text{ kipft}$$

Considering x-direction:

$M_{max} = 10.046 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.046 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.14786$$

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

Considering z-direction:

$M_{max} = 0.081173 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.081173 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.0011947$$

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

REFERENCES	CALCULATIONS	RESULTS

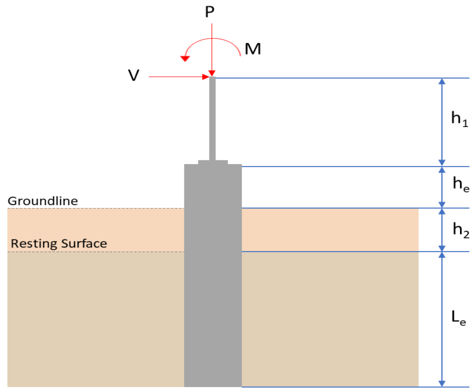
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 6.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - 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)	8.742	14.826
V_x (kip)	-0.177	-0.296
V_z (kip)	-0.112	-0.195
M_x (kipft)	-0.336	-0.585
M_z (kipft)	5.711	9.999

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.177\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.059\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(5.711\ \text{kipft}) + ((-0.177\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 1.9037\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.9094\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.112\ \text{kip})}{(36\ \text{in})}$$

$$H_o = -0.037333\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.336\ \text{kipft}) + ((-0.112\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 0.112\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.9364\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.9094\ \text{ft}), (1.9364\ \text{ft})]$$

$$L_{e,req} = 5.909\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.909\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.94544$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(8.742\ \text{kip})}{(7.0686\ \text{ft}^2)}$$

$$q = 1.2367\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.2367\ \mathrm{kip/ft^2}\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.61837$$

Status: **PASS**
Ratio: **0.620**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(36\ \mathrm{in})}$$

$$L/D = 2.0833$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.059\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 1.9037\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9037\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (1.9037\ \mathrm{kipft/ft})) + (4 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2262\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.9037\ \mathrm{kipft/ft})) + (3 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (1.9037\ \mathrm{kipft/ft})) + (2 \times (-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.25685\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (1.9037\ \mathrm{kipft/ft})) + ((-0.059\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.82966\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.2262\ \mathrm{ft})}{2}$$

$$p_a = 0.31697\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.25685\ \mathrm{kip/ft^2}\right)}{\left(0.31697\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.81032$$

Status: **PASS**
Ratio: **0.810**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.82966\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88497$$

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

Considering z-direction:

$H_o = -0.037333\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.112\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.112\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.112\ \mathrm{kipft/ft})) + (4 \times (-0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.4695\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.112\ \mathrm{kipft/ft})) + (3 \times (-0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.112\ \mathrm{kipft/ft})) + (2 \times (-0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.014656\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (0.112\ \mathrm{kipft/ft})) + ((-0.037333\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = -0.0022519\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4695\ \mathrm{ft})}{2}$$

$$p_a = 0.33521\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.014656\ \mathrm{kip/ft^2}\right)}{\left(0.33521\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.043722$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

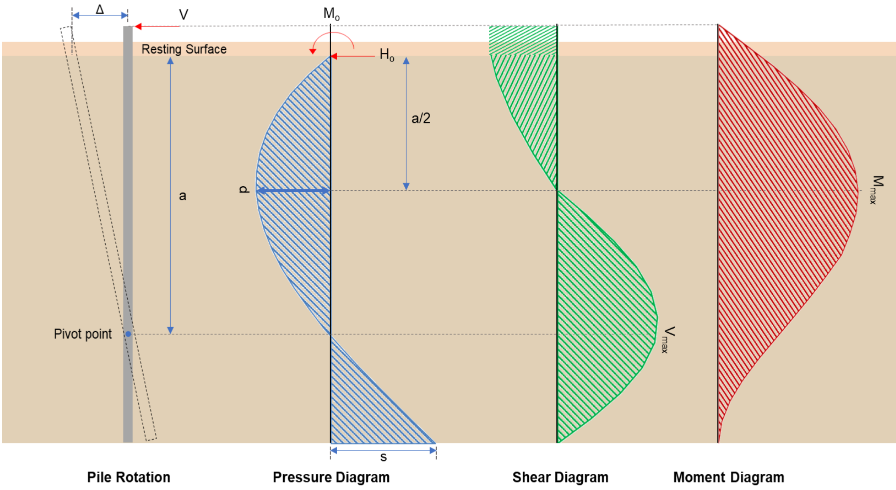
$$Ratio = \frac{s}{\ }$$

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

$$Ratio = \frac{(-0.0022519\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = -0.0024021$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.296\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.098667\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(9.999\ \mathrm{kipft}) + ((-0.296\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 3.333\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.333\ \mathrm{kipft/ft})}{(-0.098667\ \mathrm{kip/ft})}$$

$$E = 33.78\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.333\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.098667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (3.333\ \mathrm{kipft/ft})) + (4 \times (-0.098667\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.2239\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.098667\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (33.78\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.2239\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (33.78\ \mathrm{ft})}{(6.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.2239\ \mathrm{ft})}{(6.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 3.0323 \text{ kip}$$

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

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

$$M_{max} = ((-0.098667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(33.78 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2239 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (33.78 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.2239 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (33.78 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.2239 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 9.3062 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.195 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.065 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.585 \text{ kipft}) + ((-0.195 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.195 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.195 \text{ kipft/ft})}{(-0.065 \text{ kip/ft})}$$

$$E = 3 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.195 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.065 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.195 \text{ kipft/ft})) + (4 \times (-0.065 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4695 \text{ ft}$$

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

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

$$V_{max} = ((-0.065 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4695 \text{ ft})}{(6.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.29563 \text{ kip}$$

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

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

$$M_{max} = ((-0.065 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 3\right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3 \text{ ft})}{(6.25 \text{ ft})} + 2\right) \times \left(\frac{(4.4695 \text{ ft})}{2 \times (6.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.81519 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(14.826 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (3 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -44.713 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-44.713 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})\right]$$

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\ \text{ksi}) \times \left[\left(1017.9\ \text{in}^2\right) - \left(1.8408\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(1.8408\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 1492.5\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(14.826\ \text{kip})}{(1492.5\ \text{kip})}$$

$$Ratio = 0.0099337$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\ \text{in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\ \text{in})$$

$$d = 28.8\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(28.8\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\ \text{psi})} \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,max} = 203.86\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$, $P = 14.826\ \text{kip} \rightarrow 14826\ \text{lbf}$,
22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\ \text{psi})} + \frac{(14826\ \text{lbf})}{6 \times (1017.9\ \text{in}^2)}\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,a} = 84.06\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,
22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\ \text{psi})} + (0.05 \times (3000\ \text{psi}))\right] \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,b} = 237.06\ \text{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(203.86\ \text{kip}), (84.06\ \text{kip}), (237.06\ \text{kip})\right]$$

$$V_c = 84.06 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((84.06 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 79.449 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.0323 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(3.0323 \text{ kip})}{(79.449 \text{ kip})}$$

$$Ratio = 0.038167$$

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

Considering z-direction:

$V_{max} = 0.29563 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.29563 \text{ kip})}{(79.449 \text{ kip})}$$

$$Ratio = 0.003721$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

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

$$\phi M_{n,1} = 67.947kipft$$

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

$$\phi M_{n,2} = \phi \; 0.85 \; f_{ck}' \; S_m$$

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

$$\phi M_{n,2} = 632.67 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$$

$$\phi M_n = 67.947 \text{ kipft}$$

Considering x-direction:

$M_{max} = 9.3062 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.3062 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.13696$$

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

Considering z-direction:

$M_{max} = 0.81519 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.81519 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.011998$$

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