

Your Project Calculations



Project Name: USDCampusPhase1-G2

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=USDCampusPhase1-G2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=cdWpgwMyqncVpHjxrF3qp5gUEChP1L7ZKT41KvbCOF6zL5rRKNDPIZEqpTZvcZj8

Array Specification

Product:	Beam
Unique ID:	4P-19.75-8TOP-SD-57-L-5Hx11W-LG1C
Duty Classification:	SD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	11
Total Number of Modules:	55
Desired Tilt Angle:	10
Front Edge Clearance:	15
Total Array Height at Tilt:	18.25 ft
Total Frame Length:	76.25 ft
Frame Weight:	3885 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	76.63 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft
Rail Check:	PASS (46% utilized)

Support Specifications

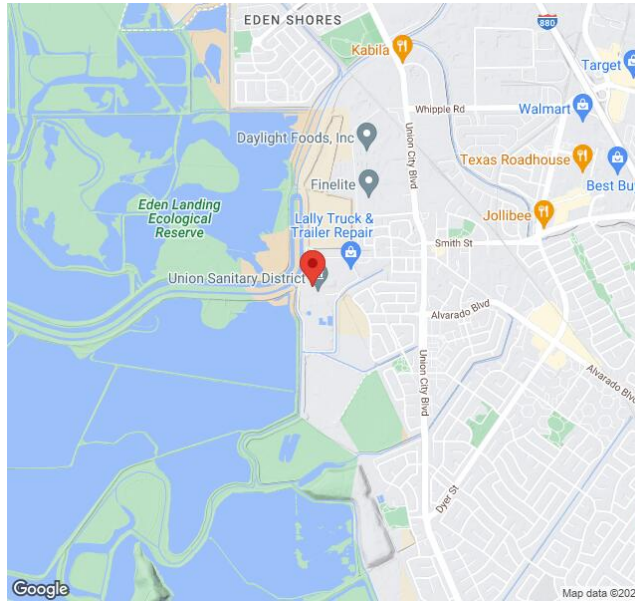
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	16.63 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 7.00 ft Pile 3: 7.00 ft Pile 4: 6.75 ft
Foundation Volume:	7.199 y ³
Foundation Result:	PASSED
Mount Twist:	0.018490 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	5072 Benson Rd, Union City, CA 94587, USA
Wind Speed:	85 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

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

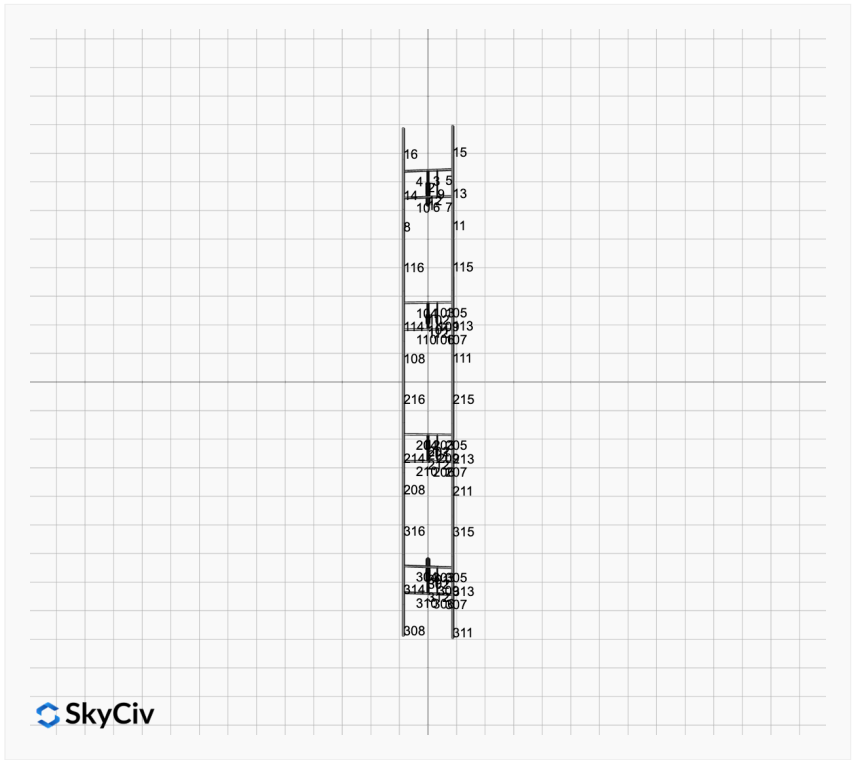
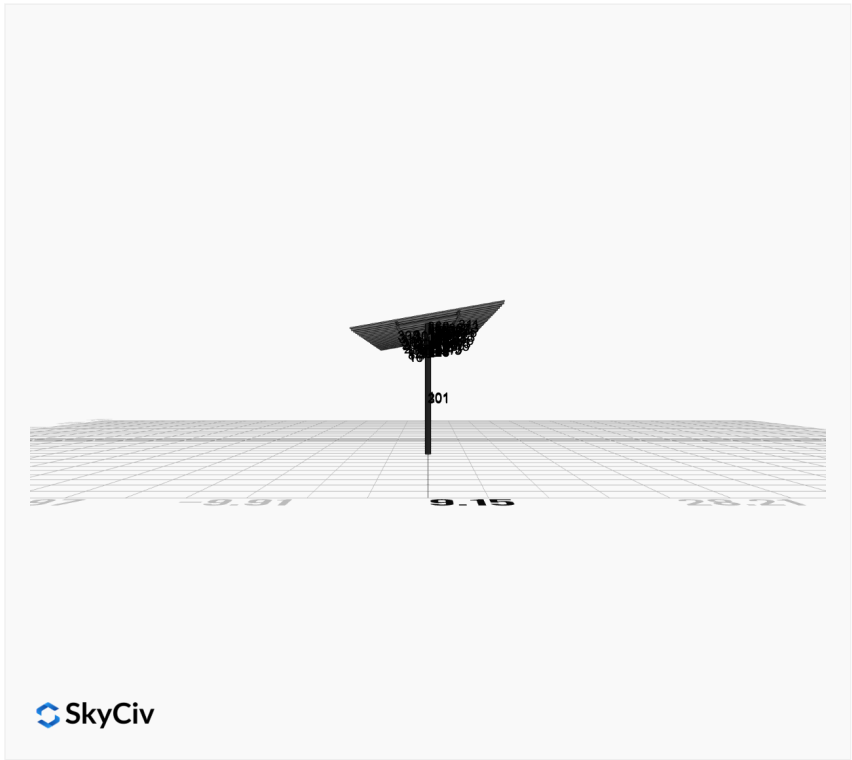
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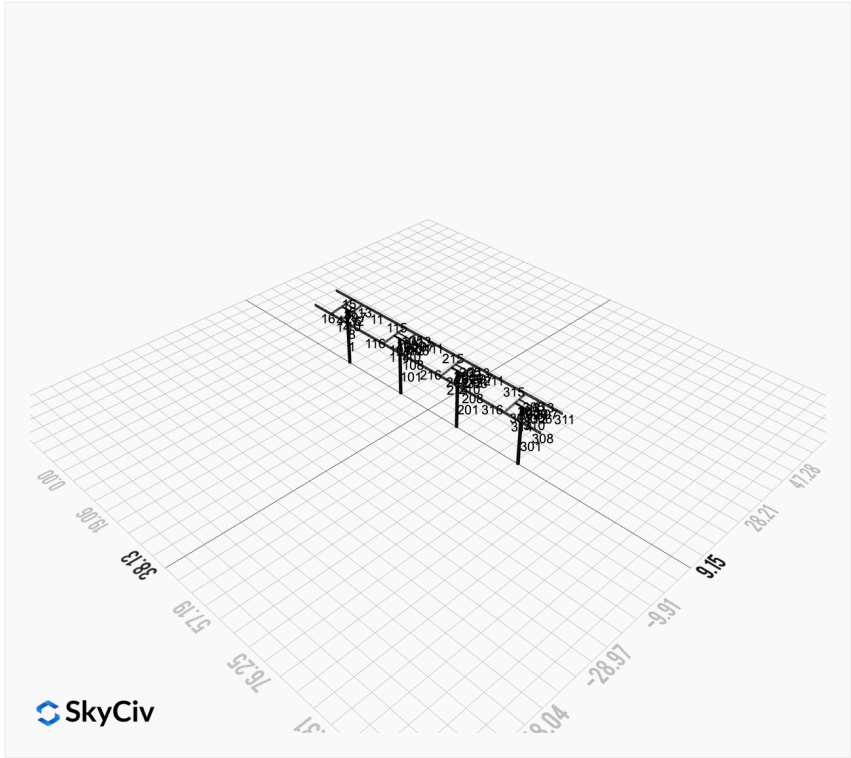
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Design Notes:

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







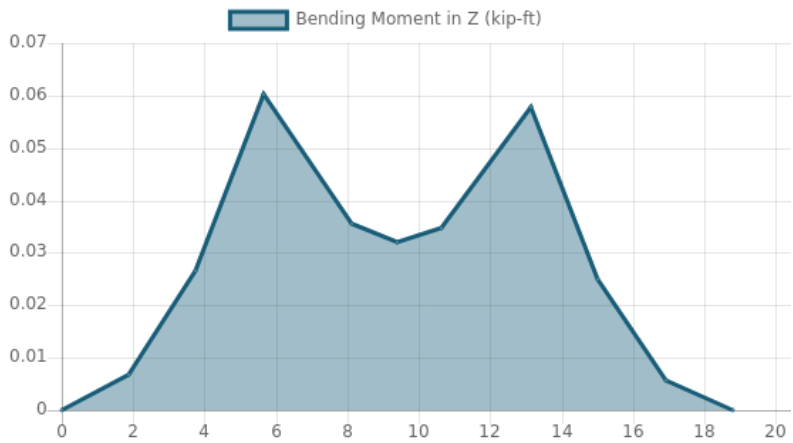
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: None
Tributary Width: 3.483333333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Wind uplift Case A (X): 0.0000 kip/ft
Wind uplift Case A (Y): 0.0303 kip/ft
Wind uplift Case A: 0.0193 kip/ft
Wind uplift Case B: 0.0193 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0432 kip/ft

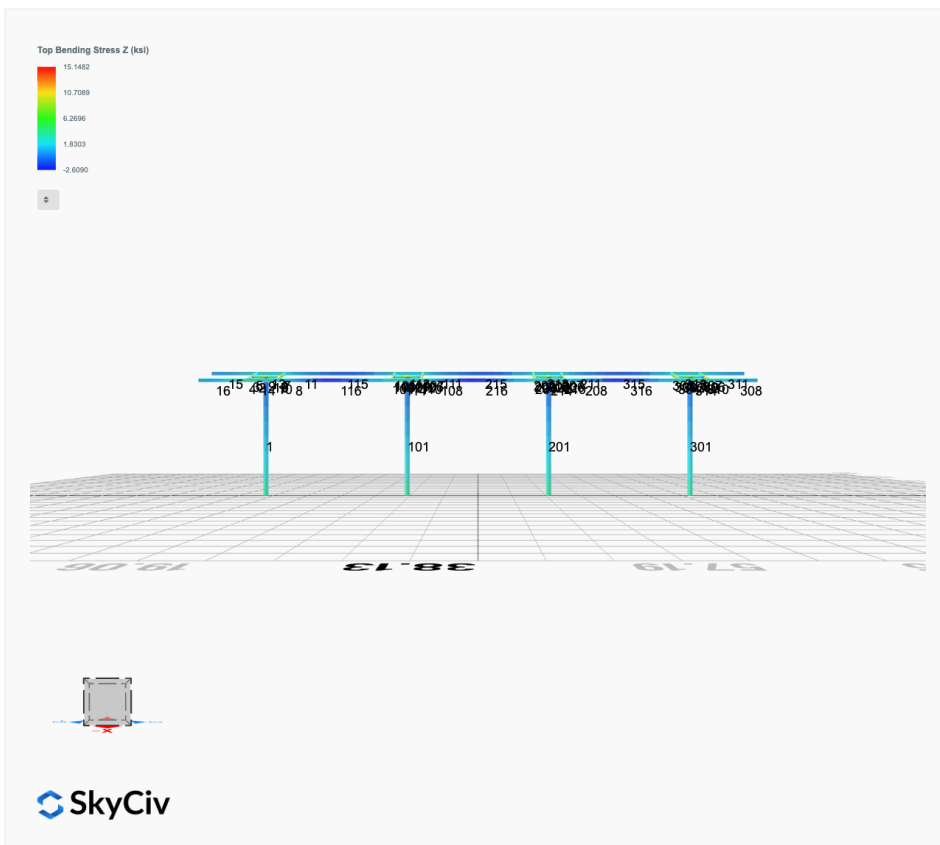
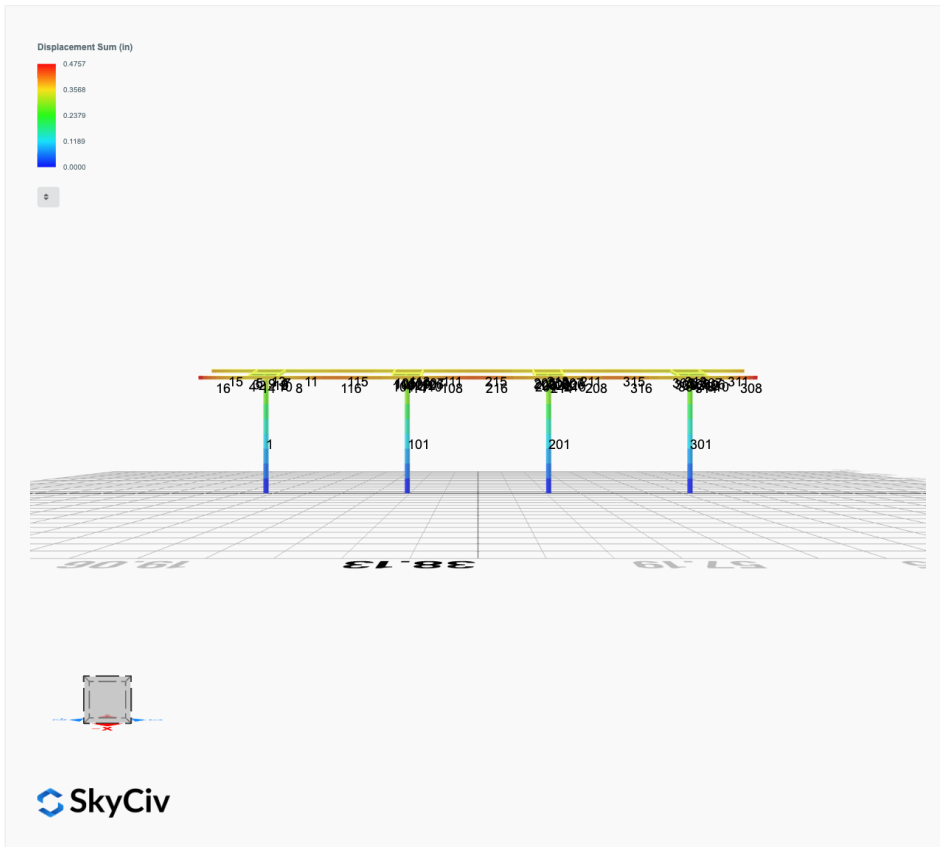


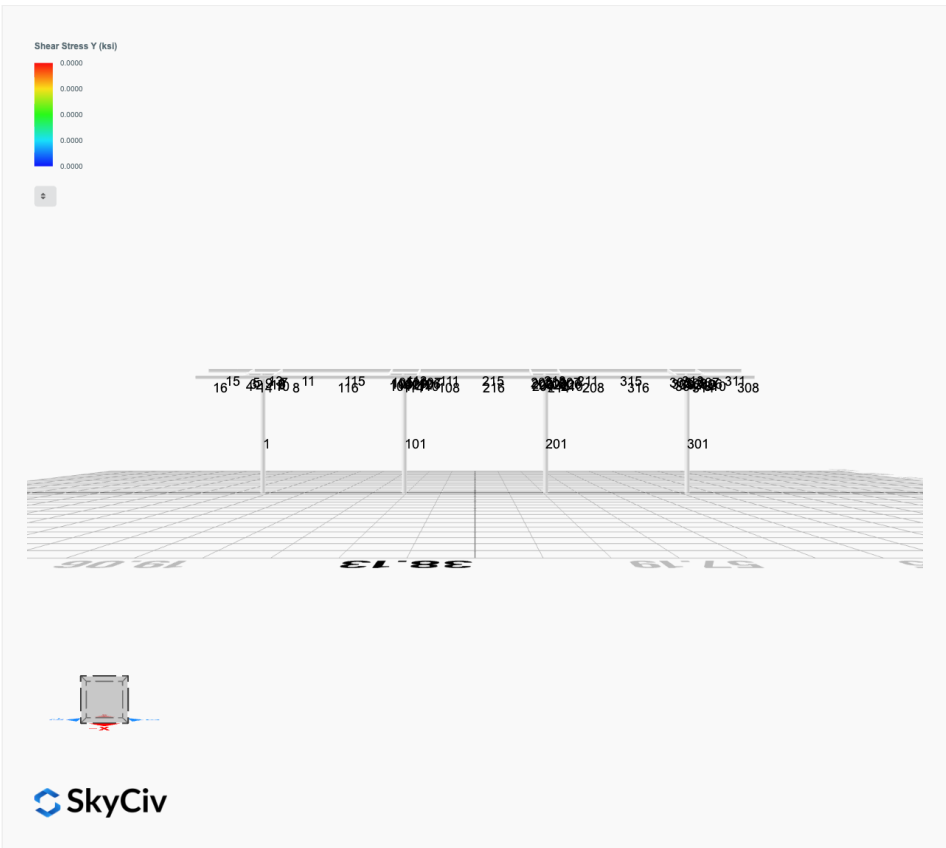
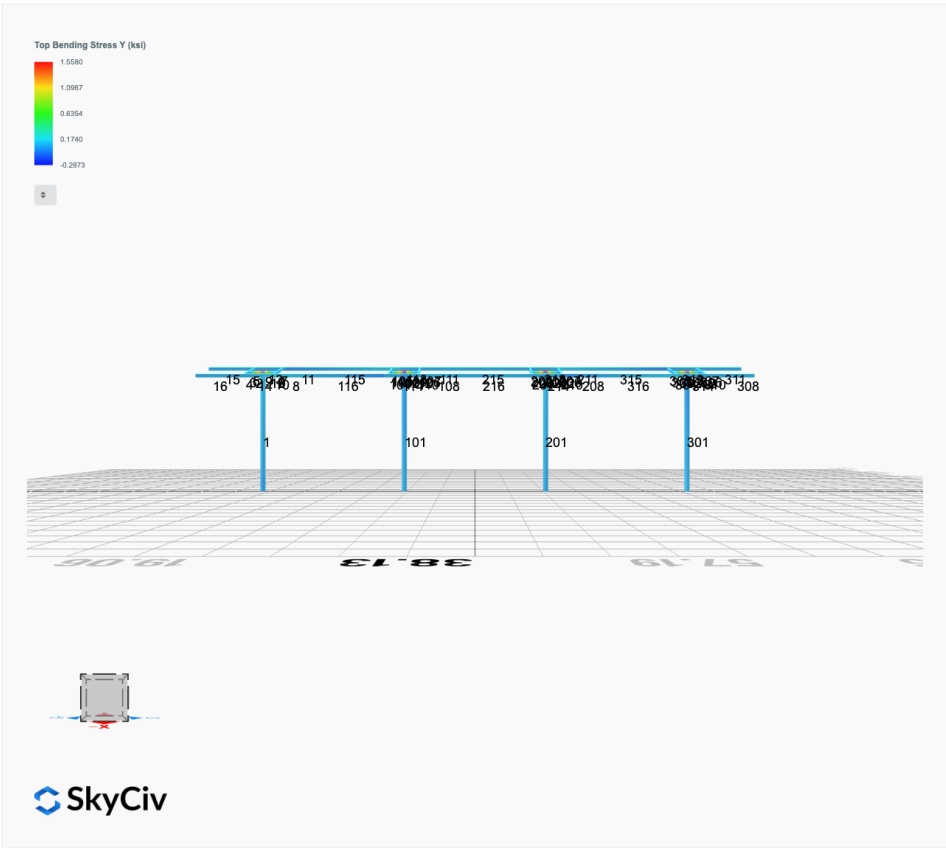
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	15.71619624	0.456	PASS
Material Yield	34.5	15.71619624	0.456	PASS
Material Strength	37	15.71619624	0.425	PASS

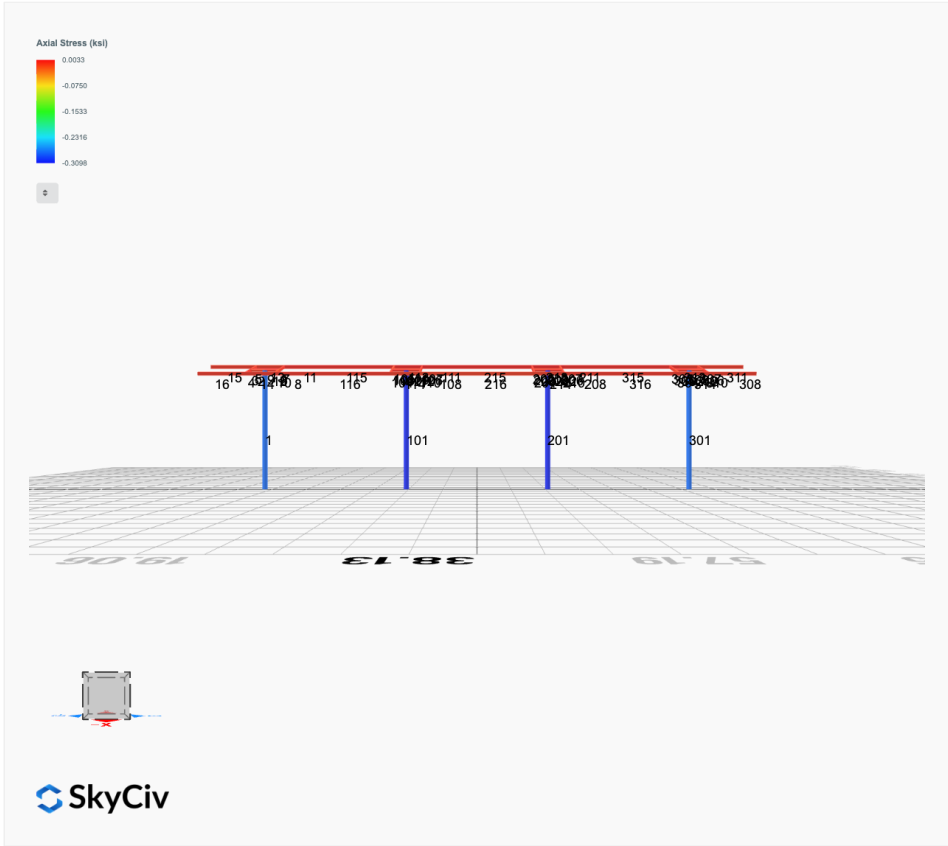
Member 1, ULS: 1. 1.4D



FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 2. D + L	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 3. D + (S or Lr or R)	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 3. D + (S or Lr or R)	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 5b. D + 0.7E	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0012	2.5145	-0.0064	-0.0331	0.0117	0.0400
ULS: 8. 0.6D + 0.7E	-0.0007	1.5087	-0.0039	-0.0199	0.0070	0.0240
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3751	4.6058	-0.0127	-0.0662	0.0129	8.0167
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3751	4.6058	-0.0127	-0.0662	0.0129	8.0167
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2589	1.0482	-0.0016	-0.0079	0.0085	-3.0068
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2301	1.2378	-0.0032	-0.0160	0.0141	-8.5400
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2816	4.0830	-0.0111	-0.0580	0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2816	4.0830	-0.0111	-0.0580	0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1939	1.4148	-0.0028	-0.0142	0.0093	-2.2451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1723	1.5570	-0.0040	-0.0203	0.0135	-6.3950
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2816	4.0830	-0.0111	-0.0580	0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2816	4.0830	-0.0111	-0.0580	0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1939	1.4148	-0.0028	-0.0142	0.0093	-2.2451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1723	1.5570	-0.0040	-0.0203	0.0135	-6.3950
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3746	3.6000	-0.0101	-0.0530	0.0083	8.0007
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3746	3.6000	-0.0101	-0.0530	0.0083	8.0007
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2594	0.0424	0.0010	0.0054	0.0038	-3.0227
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2305	0.2320	-0.0006	-0.0027	0.0095	-8.5560

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.5029
Shear X	-0.6249
Shear Z	-0.0181
Moment X	-0.0950
Moment Y (Twist)	0.0185
Moment Z	14.6121

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	4.6058
Shear X	-0.3751
Shear Z	-0.0127
Moment X	-0.0662
Moment Y (Twist)	0.0141
Moment Z	8.5560

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	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 2. D + L	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 3. D + (S or Lr or R)	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 3. D + (S or Lr or R)	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 5b. D + 0.7E	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0012	2.6380	0.0027	0.0143	-0.0023	0.0046
ULS: 8. 0.6D + 0.7E	0.0007	1.5828	0.0016	0.0086	-0.0014	0.0028
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3862	4.8641	0.0066	0.0342	-0.0104	8.3001
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3862	4.8641	0.0066	0.0342	-0.0104	8.3001
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2750	1.0766	0.0003	0.0020	0.0012	-3.1870
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2346	1.2794	-0.0000	0.0004	0.0055	-8.8676
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2893	4.3076	0.0056	0.0292	-0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2893	4.3076	0.0056	0.0292	-0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2065	1.4669	0.0009	0.0050	0.0003	-2.3891
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1762	1.6191	0.0007	0.0038	0.0035	-6.6496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2893	4.3076	0.0056	0.0292	-0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2893	4.3076	0.0056	0.0292	-0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2065	1.4669	0.0009	0.0050	0.0003	-2.3891
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1762	1.6191	0.0007	0.0038	0.0035	-6.6496
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3866	3.8089	0.0055	0.0285	-0.0095	8.2982
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3866	3.8089	0.0055	0.0285	-0.0095	8.2982
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2745	0.0214	-0.0007	-0.0038	0.0021	-3.1888
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2341	0.2243	-0.0011	-0.0054	0.0064	-8.8695

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.8757
Shear X	-0.6455
Shear Z	0.0097
Moment X	0.0507
Moment Y (Twist)	0.0166
Moment Z	15.1597

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	4.8641
Shear X	-0.3866
Shear Z	0.0066
Moment X	0.0342
Moment Y (Twist)	0.0104
Moment Z	8.8695

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	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 2. D + L	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 3. D + (S or Lr or R)	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 3. D + (S or Lr or R)	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 5b. D + 0.7E	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0012	2.6380	-0.0027	-0.0143	0.0023	0.0046
ULS: 8. 0.6D + 0.7E	0.0007	1.5828	-0.0016	-0.0086	0.0014	0.0028
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3862	4.8641	-0.0066	-0.0342	0.0104	8.3001
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3862	4.8641	-0.0066	-0.0342	0.0104	8.3001
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2750	1.0766	-0.0003	-0.0020	-0.0012	-3.1870
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2346	1.2794	0.0000	-0.0004	-0.0055	-8.8676
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2893	4.3076	-0.0056	-0.0292	0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2893	4.3076	-0.0056	-0.0292	0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2065	1.4669	-0.0009	-0.0050	-0.0003	-2.3891
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1762	1.6191	-0.0007	-0.0038	-0.0035	-6.6496

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	-0.2893	4.3076	-0.0056	-0.0292	0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2893	4.3076	-0.0056	-0.0292	0.0084	6.2262
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2065	1.4669	-0.0009	-0.0050	-0.0003	-2.3891
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1762	1.6191	-0.0007	-0.0038	-0.0035	-6.6496
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3866	3.8089	-0.0055	-0.0285	0.0095	8.2982
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3866	3.8089	-0.0055	-0.0285	0.0095	8.2982
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2745	0.0214	0.0007	0.0038	-0.0021	-3.1888
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2341	0.2243	0.0011	0.0054	-0.0064	-8.8695

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.8757
Shear X	-0.6455
Shear Z	-0.0097
Moment X	-0.0507
Moment Y (Twist)	0.0166
Moment Z	15.1597

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	4.8641
Shear X	-0.3866
Shear Z	-0.0066
Moment X	-0.0342
Moment Y (Twist)	0.0104
Moment Z	8.8695

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	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 2. D + L	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 3. D + (S or Lr or R)	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 3. D + (S or Lr or R)	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 5b. D + 0.7E	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0012	2.5145	0.0064	0.0332	-0.0117	0.0400
ULS: 8. 0.6D + 0.7E	-0.0007	1.5087	0.0039	0.0199	-0.0070	0.0240
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3751	4.6058	0.0127	0.0662	-0.0129	8.0167
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3751	4.6058	0.0127	0.0662	-0.0129	8.0167
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2589	1.0482	0.0016	0.0079	-0.0085	-3.0068
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2301	1.2378	0.0032	0.0160	-0.0141	-8.5400
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2816	4.0830	0.0111	0.0580	-0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2816	4.0830	0.0111	0.0580	-0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1939	1.4148	0.0028	0.0142	-0.0093	-2.2451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1723	1.5570	0.0040	0.0203	-0.0135	-6.3950
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2816	4.0830	0.0111	0.0580	-0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2816	4.0830	0.0111	0.0580	-0.0126	6.0225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1939	1.4148	0.0028	0.0142	-0.0093	-2.2451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1723	1.5570	0.0040	0.0203	-0.0135	-6.3950
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3746	3.6000	0.0101	0.0530	-0.0083	8.0007
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3746	3.6000	0.0101	0.0530	-0.0083	8.0007
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2594	0.0424	-0.0010	-0.0054	-0.0038	-3.0227
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2305	0.2320	0.0006	0.0027	-0.0095	-8.5560

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

Result	Value (kip, kip-ft)
Axial	6.5029
Shear X	-0.6249
Shear Z	0.0181
Moment X	0.0950
Moment Y (Twist)	0.0185
Moment Z	14.6122

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	4.6058
Shear X	-0.3751
Shear Z	0.0127
Moment X	0.0662
Moment Y (Twist)	0.0141
Moment Z	8.5560

Project Details

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

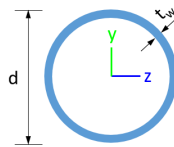


Design Input Information

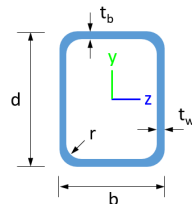
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

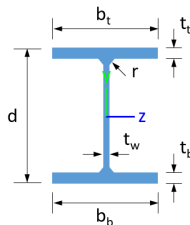
Section Dimensions



ID	Name	d (in)	t_w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
9	8in Pipe Sch 40	8.63	0.32				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

302	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
303	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.21,1.17,1.18,1.18,1.21,1.17,1.18,1.18,1.19,1.31,1.1 8,1.18,1.19,1.31,1.18,1.18,1.13,1.17	3 0 0	2 0 0	1
304	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.70,1.68,1.6 7,1.67,1.70,1.68,1.67,1.67,1.65,1.68	3 0 0	2 0 0	1
305	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.71,1.66,1.67,1.67,1.71,1.66,1.67,1.67,1.68,1.92,1.6 7,1.67,1.68,1.92,1.67,1.67,1.62,1.66	3 0 0	2 0 0	1
306	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.22,1.17,1.18,1.18,1.22,1.17,1.18,1.18,1.19,1.28,1.1 8,1.18,1.19,1.28,1.18,1.18,1.13,1.17	3 0 0	2 0 0	1
307	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.71,1.66,1.67,1.67,1.71,1.66,1.67,1.67,1.68,1.81,1.6 7,1.67,1.68,1.81,1.67,1.67,1.62,1.66	3 0 0	2 0 0	1
308	18	9.97	9.97	4.7 5	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3 3,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1
309	1	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
310	15	2.44	2.44	3.7 5	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.70,1.68,1.6 7,1.67,1.70,1.68,1.67,1.67,1.65,1.68	3 0 0	2 0 0	1
311	18	9.97	9.97	4.7 5	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3 3,2.33,2.33,2.33,2.33,2.33,2.33	3 0 0	2 0 0	1
312	4	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
313	18	4.88	4.00	7.5 0	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.24,1.16,1.10,1.10,1.24,1.16,1.10,1.10,1.10,1.11,1.1 0,1.10,1.10,1.11,1.10,1.10,1.09,1.14	3 0 0	2 0 0	1
314	18	4.88	4.00	7.5 0	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.14,1.14,1.10,1.10,1.14,1.14,1.10,1.10,1.10,1.10,1.1 0,1.10,1.10,1.10,1.10,1.10,1.10,1.18	3 0 0	2 0 0	1
315	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.09,1.1 7,1.17,1.17,1.09,1.17,1.17,1.16,1.18	3 0 0	2 0 0	1
316	18	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.18,1.17,1.17,1.17,1.17,1.1 7,1.17,1.17,1.17,1.17,1.17,1.17,1.18	3 0 0	2 0 0	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	93.23	83.29	83.29	113.39	113.39
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.19	6.45	30.09	45.74
14	120.60	98.23	19.37	6.45	30.09	45.74
15	120.60	34.69	23.36	6.45	30.09	45.74
16	120.60	34.69	23.36	6.45	30.09	45.74
101	377.97	93.23	83.29	83.29	113.39	113.39
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61

103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	17.78	6.45	30.09	45.74
114	120.60	98.23	18.31	6.45	30.09	45.74
115	120.60	68.63	15.57	6.45	30.09	45.74
116	120.60	68.63	15.71	6.45	30.09	45.74
201	377.97	93.23	83.29	83.29	113.39	113.39
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	117.88	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	117.88	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	17.78	6.45	30.09	45.74
214	120.60	98.23	18.31	6.45	30.09	45.74
215	120.60	68.63	15.57	6.45	30.09	45.74
216	120.60	68.63	15.71	6.45	30.09	45.74
301	377.97	93.23	83.29	83.29	113.39	113.39
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	4.60	29.14	16.61
304	79.65	72.01	10.99	4.60	29.14	16.61
305	79.65	73.44	10.99	4.60	29.14	16.61
306	79.65	74.02	10.99	4.60	29.14	16.61
307	79.65	73.44	10.99	4.60	29.14	16.61
308	120.60	34.69	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	4.60	29.14	16.61
311	120.60	34.69	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	98.23	19.19	6.45	30.09	45.74
314	120.60	98.23	19.37	6.45	30.09	45.74
315	120.60	68.63	14.89	6.45	30.09	45.74
316	120.60	68.63	15.98	6.45	30.09	45.74

Design Ratio

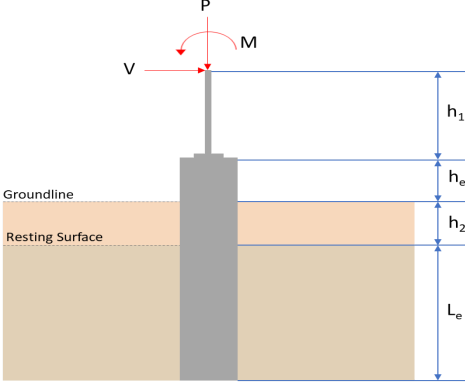
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.070	0.175	0.002	0.006	0.000	0.200	#13	0.713	Not Required	Pass
2	0.000	0.339	0.043	0.070	0.007	0.383	#13	0.034	Not Required	Pass
3	0.002	0.564	0.016	0.057	0.002	0.580	#13	0.044	Not Required	Pass
4	0.001	0.445	0.027	0.045	0.005	0.465	#13	0.078	Not Required	Pass

4	0.001	0.445	0.057	0.045	0.005	0.405	#13	0.078	Not Required	Pass
5	0.002	0.349	0.041	0.057	0.006	0.357	#13	0.073	Not Required	Pass
6	0.002	0.553	0.015	0.056	0.001	0.565	#13	0.044	Not Required	Pass
7	0.002	0.343	0.041	0.056	0.006	0.350	#13	0.073	Not Required	Pass
8	0.000	0.033	0.017	0.028	0.002	0.041	#13	0.059	Not Required	Pass
9	0.002	0.063	0.013	0.001	0.000	0.077	#13	0.198	Not Required	Pass
10	0.002	0.432	0.040	0.044	0.005	0.456	#13	0.078	Not Required	Pass
11	0.000	0.041	0.017	0.036	0.002	0.049	#13	0.088	Not Required	Pass
12	0.000	0.326	0.042	0.068	0.007	0.369	#13	0.034	Not Required	Pass
13	0.001	0.196	0.041	0.046	0.002	0.226	#13	0.265	Not Required	Pass
14	0.001	0.160	0.041	0.036	0.002	0.185	#13	0.177	Not Required	Pass
15	0.000	0.085	0.022	0.028	0.001	0.103	#13	Not Required	Not Required	Pass
16	0.000	0.067	0.022	0.022	0.001	0.085	#13	Not Required	Not Required	Pass
101	0.074	0.182	0.001	0.006	0.000	0.207	#13	0.713	Not Required	Pass
102	0.000	0.346	0.044	0.073	0.007	0.390	#13	0.034	Not Required	Pass
103	0.002	0.589	0.017	0.060	0.002	0.605	#13	0.044	Not Required	Pass
104	0.002	0.464	0.038	0.047	0.005	0.492	#13	0.078	Not Required	Pass
105	0.002	0.365	0.038	0.059	0.005	0.373	#13	0.073	Not Required	Pass
106	0.002	0.597	0.018	0.061	0.002	0.612	#13	0.044	Not Required	Pass
107	0.002	0.370	0.039	0.060	0.006	0.379	#13	0.073	Not Required	Pass
108	0.000	0.038	0.014	0.028	0.002	0.044	#13	0.088	Not Required	Pass
109	0.001	0.061	0.012	0.001	0.000	0.073	#13	0.198	Not Required	Pass
110	0.002	0.470	0.037	0.048	0.005	0.491	#13	0.078	Not Required	Pass
111	0.000	0.049	0.014	0.036	0.002	0.054	#13	0.088	Not Required	Pass
112	0.000	0.353	0.045	0.074	0.008	0.398	#13	0.034	Not Required	Pass
113	0.001	0.163	0.038	0.046	0.002	0.190	#13	0.265	Not Required	Pass
114	0.001	0.136	0.038	0.036	0.002	0.159	#13	0.177	Not Required	Pass
115	0.000	0.157	0.022	0.035	0.002	0.176	#13	0.439	Not Required	Pass
116	0.000	0.125	0.022	0.028	0.002	0.144	#13	0.293	Not Required	Pass
201	0.074	0.182	0.001	0.006	0.000	0.207	#13	0.713	Not Required	Pass
202	0.000	0.353	0.045	0.074	0.008	0.398	#13	0.034	Not Required	Pass
203	0.002	0.597	0.018	0.061	0.002	0.612	#13	0.044	Not Required	Pass
204	0.002	0.470	0.037	0.048	0.005	0.491	#13	0.078	Not Required	Pass
205	0.002	0.370	0.039	0.060	0.006	0.379	#13	0.073	Not Required	Pass
206	0.002	0.589	0.017	0.060	0.002	0.605	#13	0.044	Not Required	Pass
207	0.002	0.365	0.038	0.059	0.005	0.373	#13	0.073	Not Required	Pass
208	0.000	0.037	0.016	0.028	0.002	0.040	#13	0.059	Not Required	Pass
209	0.001	0.061	0.012	0.001	0.000	0.073	#13	0.198	Not Required	Pass
210	0.002	0.464	0.038	0.047	0.005	0.492	#13	0.078	Not Required	Pass
211	0.000	0.047	0.016	0.035	0.002	0.050	#13	0.088	Not Required	Pass
212	0.000	0.346	0.044	0.073	0.007	0.390	#13	0.034	Not Required	Pass
213	0.001	0.163	0.038	0.046	0.002	0.190	#13	0.265	Not Required	Pass
214	0.001	0.136	0.038	0.036	0.002	0.159	#13	0.265	Not Required	Pass
215	0.000	0.165	0.022	0.036	0.002	0.183	#13	0.439	Not Required	Pass
216	0.000	0.130	0.022	0.028	0.002	0.149	#13	0.439	Not Required	Pass
301	0.070	0.175	0.002	0.006	0.000	0.200	#13	0.713	Not Required	Pass
302	0.000	0.326	0.042	0.068	0.007	0.369	#13	0.034	Not Required	Pass
303	0.002	0.553	0.015	0.056	0.001	0.565	#13	0.044	Not Required	Pass
304	0.002	0.432	0.040	0.044	0.005	0.456	#13	0.078	Not Required	Pass
305	0.002	0.343	0.041	0.056	0.006	0.350	#13	0.073	Not Required	Pass
306	0.002	0.564	0.016	0.057	0.002	0.580	#13	0.044	Not Required	Pass
307	0.002	0.349	0.041	0.057	0.006	0.357	#13	0.073	Not Required	Pass
308	0.000	0.067	0.022	0.022	0.001	0.085	#13	Not Required	Not Required	Pass
309	0.002	0.063	0.013	0.001	0.000	0.077	#13	0.198	Not Required	Pass

310	0.001	0.445	0.037	0.045	0.005	0.465	#13	0.078	Not Required	Pass
311	0.000	0.085	0.022	0.028	0.001	0.103	#13	Not Required	Not Required	Pass
312	0.000	0.339	0.043	0.070	0.007	0.383	#13	0.034	Not Required	Pass
313	0.001	0.196	0.041	0.046	0.002	0.226	#13	0.177	Not Required	Pass
314	0.001	0.160	0.041	0.036	0.002	0.185	#13	0.177	Not Required	Pass
315	0.000	0.155	0.022	0.036	0.002	0.174	#13	0.439	Not Required	Pass
316	0.000	0.123	0.022	0.028	0.002	0.142	#13	0.293	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 6.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.606</td> <td>6.503</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.375</td> <td>-0.625</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.013</td> <td>-0.018</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.066</td> <td>-0.095</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.556</td> <td>14.612</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.606	6.503	V_x (kip)	-0.375	-0.625	V_z (kip)	-0.013	-0.018	M_x (kipft)	-0.066	-0.095	M_z (kipft)	8.556	14.612	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.375 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.125 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.556 \text{ kipft}) + ((-0.375 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.852 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.5515 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.013 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0043333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.066 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.022 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3067 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.5515 \text{ ft}), (1.3067 \text{ ft})]$$

$$L_{e,req} = 6.551 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.551 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.97052$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.606 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.65162 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.65162 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32581$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.125 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.852 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.852 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.852 \text{ kipft/ft})) + (4 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5927 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.852 \text{ kipft/ft})) + (3 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.852 \text{ kipft/ft})) + (2 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.29661 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.852 \text{ kipft/ft})) + ((-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 1.0054 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5927 \text{ ft})}{2}$$

$$p_a = 0.34445 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29661 \text{ kip/ft}^2)}{(0.34445 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86111$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0054 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99297$$

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

Considering z-direction:

$H_o = -0.0043333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.022 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.022 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.022 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7643 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.022 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.022 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 215.45 \times 10^{-9} \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.022 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.0030512 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7643 \text{ ft})}{2}$$

$$p_a = 0.35732 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(215.45 \times 10^{-9} \text{ kip/ft}^2)}{(0.35732 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 602.96 \times 10^{-9}$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

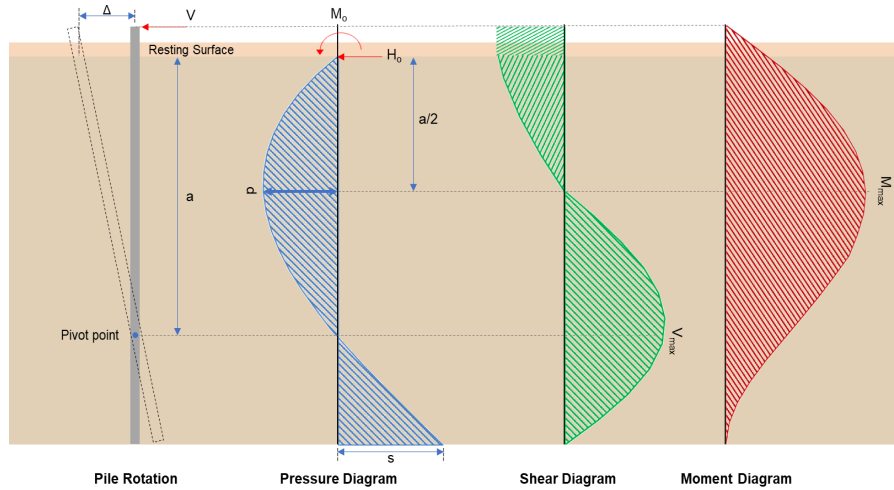
Status: **PASS**
Ratio: **0.000**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0030512 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0030135$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.625 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.20833 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.612 \text{ kipft}) + ((-0.625 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.8707 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.8707 \text{ kipft/ft})}{(-0.20833 \text{ kip/ft})}$$

$$E = 23.379 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8707 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.20833 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.8707 \text{ kipft/ft})) + (4 \times (-0.20833 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5908 \text{ ft}$$

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

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

$$V_{max} = ((-0.20833 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5908 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5908 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2476 \text{ kip}$$

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

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

$$M_{max} = ((-0.20833 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(23.379 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.95 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.018 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.095 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.031667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.031667 \text{ kipft/ft})}{(-0.006 \text{ kip/ft})}$$

$$E = 5.2778 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.031667 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.006 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.031667 \text{ kipft/ft})) + (4 \times (-0.006 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7589 \text{ ft}$$

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

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

$$V_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7589 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7589 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.036823 \text{ kip}$$

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

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

$$M_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(5.2778 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.11337 \text{ kipft}$$

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,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.503 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.975 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.975 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

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

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

Summary:

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.503 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0043572$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

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

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

$$\lambda_s = 0.71796$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.503 \text{ kip} \rightarrow 6503 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6503 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.647 \text{ kip}$$

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)

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

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

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

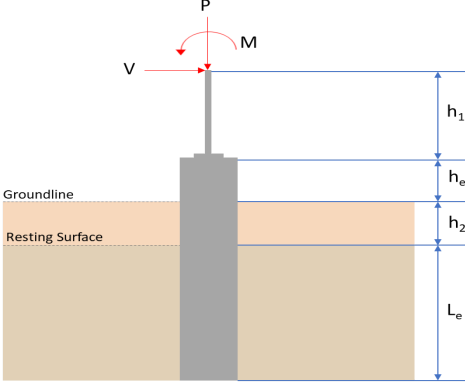
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(203.86 \text{ kip}), (82.647 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.647 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.647 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.531 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.2476 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.2476 \text{ kip})}{(78.531 \text{ kip})}$ $Ratio = 0.054088$ <p>Considering z-direction:</p> <p>$V_{max} = 0.036823 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.036823 \text{ kip})}{(78.531 \text{ kip})}$ $Ratio = 0.0004689$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4500.4 \text{ in}^3$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 13.95 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(13.95 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.2053$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.11337 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.11337 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.0016685$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 6.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.606</td> <td>6.503</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.375</td> <td>-0.625</td> </tr> <tr> <td>V_z (kip)</td> <td>0.013</td> <td>0.018</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.066</td> <td>0.095</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.556</td> <td>14.612</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.606	6.503	V_x (kip)	-0.375	-0.625	V_z (kip)	0.013	0.018	M_x (kipft)	0.066	0.095	M_z (kipft)	8.556	14.612	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.375 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.125 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.556 \text{ kipft}) + ((-0.375 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.852 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.5515 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.013 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0043333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.066 \text{ kipft}) + ((0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.022 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5003 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.5515 \text{ ft}), (1.5003 \text{ ft})]$$

$$L_{e,req} = 6.551 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.551 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.97052$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.606 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.65162 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.65162 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32581$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.125 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.852 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.852 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.852 \text{ kipft/ft})) + (4 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5927 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.852 \text{ kipft/ft})) + (3 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.852 \text{ kipft/ft})) + (2 \times (-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.29661 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.852 \text{ kipft/ft})) + ((-0.125 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 1.0054 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5927 \text{ ft})}{2}$$

$$p_a = 0.34445 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29661 \text{ kip/ft}^2)}{(0.34445 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86111$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0054 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99297$$

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

Considering z-direction:

$H_o = 0.0043333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.022 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.022 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.022 \text{ kipft/ft})) + (4 \times (0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7643 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.022 \text{ kipft/ft})) + (3 \times (0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.022 \text{ kipft/ft})) + (2 \times (0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.0064144 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.022 \text{ kipft/ft})) + ((0.0043333 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.015152 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7643 \text{ ft})}{2}$$

$$p_a = 0.35732 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0064144 \text{ kip/ft}^2)}{(0.35732 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.017951$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

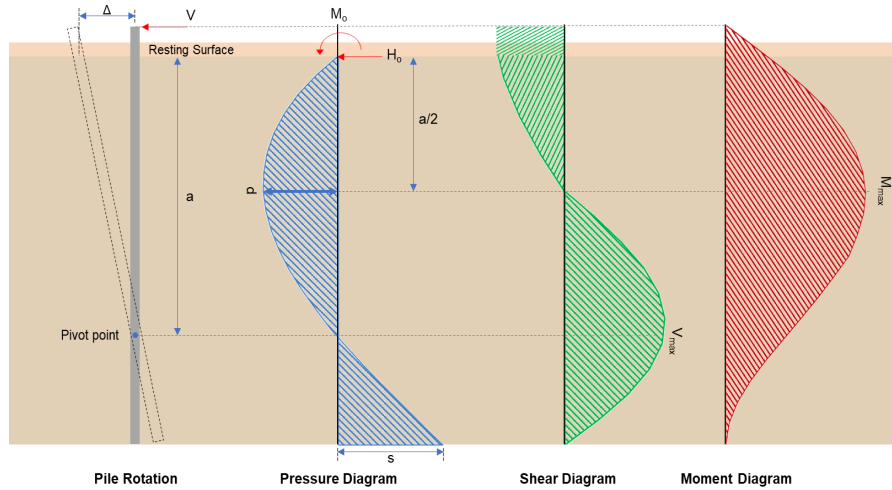
Status: **PASS**
Ratio: **0.020**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.015152 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.014965$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.625 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.20833 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.612 \text{ kipft}) + ((-0.625 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.8707 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.8707 \text{ kipft/ft})}{(-0.20833 \text{ kip/ft})}$$

$$E = 23.379 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.8707 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.20833 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.8707 \text{ kipft/ft})) + (4 \times (-0.20833 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5908 \text{ ft}$$

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

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

$$V_{max} = ((-0.20833 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5908 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5908 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2476 \text{ kip}$$

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

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

$$M_{max} = ((-0.20833 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(23.379 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.379 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.5908 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.95 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.018 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.006 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.095 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.031667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.031667 \text{ kipft/ft})}{(0.006 \text{ kip/ft})}$$

$$E = 5.2778 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.031667 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.006 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.031667 \text{ kipft/ft})) + (4 \times (0.006 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7589 \text{ ft}$$

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

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

$$V_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7589 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7589 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.036823 \text{ kip}$$

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

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

$$M_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(5.2778 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2778 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7589 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.11337 \text{ kipft}$$

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,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(6.503 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.975 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-44.975 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$Ratio = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

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

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

Summary:

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.503 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0043572$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

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

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

$$\lambda_s = 0.71796$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.503 \text{ kip} \rightarrow 6503 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6503 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.647 \text{ kip}$$

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)

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

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

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

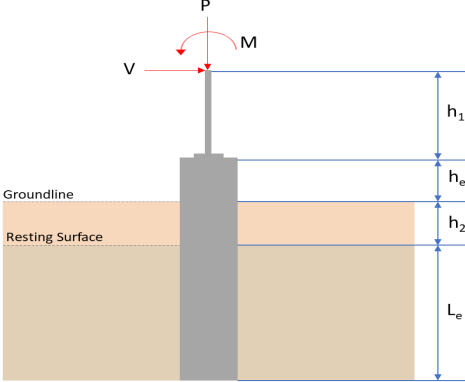
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.647 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.647 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.647 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.531 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.2476 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.2476 \text{ kip})}{(78.531 \text{ kip})}$ $Ratio = 0.054088$ <p>Considering z-direction:</p> <p>$V_{max} = 0.036823 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.036823 \text{ kip})}{(78.531 \text{ kip})}$ $Ratio = 0.0004689$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4500.4 \text{ in}^3$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 13.95 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(13.95 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.2053$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.11337 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.11337 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.0016685$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 7$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.864</td> <td>6.876</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.387</td> <td>-0.646</td> </tr> <tr> <td>V_z (kip)</td> <td>0.007</td> <td>0.010</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.034</td> <td>0.051</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.869</td> <td>15.160</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.864	6.876	V_x (kip)	-0.387	-0.646	V_z (kip)	0.007	0.010	M_x (kipft)	0.034	0.051	M_z (kipft)	8.869	15.160	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.387 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.129 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.869 \text{ kipft}) + ((-0.387 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.9563 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.6262 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.007 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0023333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034 \text{ kipft}) + ((0.007 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

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

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:

$L_{e,z} = 1.19 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.6262 \text{ ft}), (1.19 \text{ ft})]$$

$$L_{e,req} = 6.626 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.626 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.94657$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.864 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.68812 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.68812 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.34406$$

Status: **PASS**
Ratio: **0.340**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.129 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9563 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9563 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.9563 \text{ kipft/ft})) + (4 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7654 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.9563 \text{ kipft/ft})) + (3 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.9563 \text{ kipft/ft})) + (2 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.28288 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.9563 \text{ kipft/ft})) + ((-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.96359 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7654 \text{ ft})}{2}$$

$$p_a = 0.3574 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28288 \text{ kip/ft}^2)}{(0.3574 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79149$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.96359 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91771$$

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

Considering z-direction:

$H_o = 0.0023333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.0023333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.011333 \text{ kipft/ft})) + (4 \times (0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9525 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011333 \text{ kipft/ft})) + (3 \times (0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.011333 \text{ kipft/ft})) + (2 \times (0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.003209 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.011333 \text{ kipft/ft})) + ((0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.0075015 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9525 \text{ ft})}{2}$$

$$p_a = 0.37144 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.003209 \text{ kip/ft}^2)}{(0.37144 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0086394$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

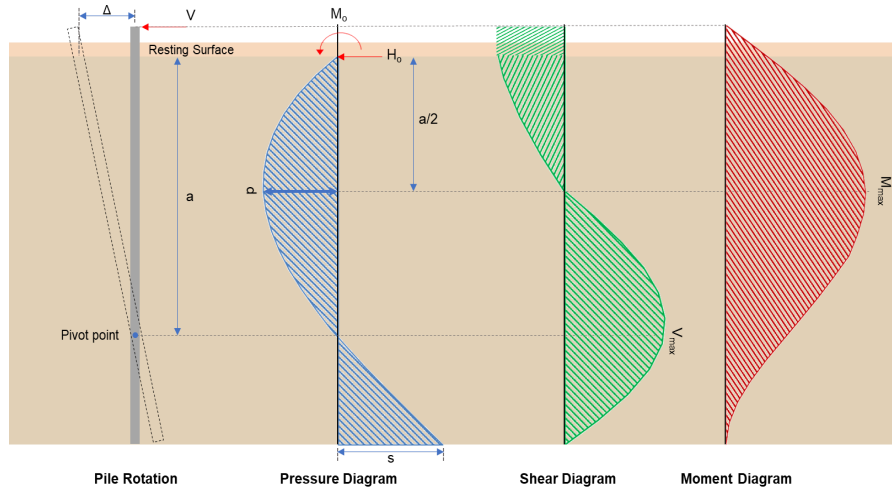
Status: **PASS**
Ratio: **0.010**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0075015 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0071443$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.646 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.21533 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.16 \text{ kipft}) + ((-0.646 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.0533 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0533 \text{ kipft/ft})}{(-0.21533 \text{ kip/ft})}$$

$$E = 23.467 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0533 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.21533 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.0533 \text{ kipft/ft})) + (4 \times (-0.21533 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7634 \text{ ft}$$

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

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

$$V_{max} = ((-0.21533 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2629 \text{ kip}$$

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

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

$$M_{max} = ((-0.21533 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.467 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 14.506 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.01 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.0033333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.051 \text{ kipft}) + ((0.01 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.017 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017 \text{ kipft/ft})}{(0.0033333 \text{ kip/ft})}$$

$$E = 5.1 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.0033333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.017 \text{ kipft/ft})) + (4 \times (0.0033333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9454 \text{ ft}$$

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

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

$$V_{max} = ((0.0033333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9454 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9454 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019519 \text{ kip}$$

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

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

$$M_{max} = ((0.0033333 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5.1 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.062041 \text{ kipft}$$

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,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.876 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.963 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.963 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

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

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

Summary:

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.876 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0046071$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

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

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

$$\lambda_s = 0.71796$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.876 \text{ kip} \rightarrow 6876 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6876 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.71 \text{ kip}$$

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)

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

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

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

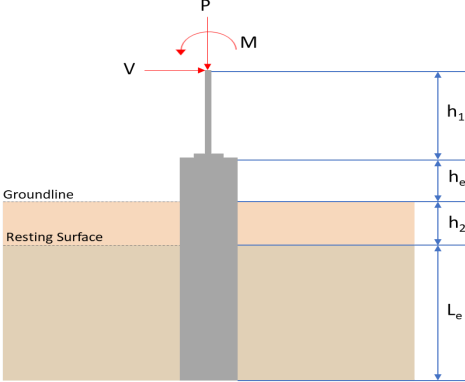
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(203.86 \text{ kip}), (82.71 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.71 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.71 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.572 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.2629 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.2629 \text{ kip})}{(78.572 \text{ kip})}$ $Ratio = 0.054254$ <p>Considering z-direction:</p> <p>$V_{max} = 0.019519 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.019519 \text{ kip})}{(78.572 \text{ kip})}$ $Ratio = 0.00024843$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 14.506 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.506 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.2135$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.062041 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.062041 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.00091308$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 7$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.864</td> <td>6.876</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.387</td> <td>-0.646</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.007</td> <td>-0.010</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.034</td> <td>-0.051</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.869</td> <td>15.160</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.864	6.876	V_x (kip)	-0.387	-0.646	V_z (kip)	-0.007	-0.010	M_x (kipft)	-0.034	-0.051	M_z (kipft)	8.869	15.160	
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M_z (kipft)	8.869	15.160																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.387 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.129 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.869 \text{ kipft}) + ((-0.387 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.9563 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.6262 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.007 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0023333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034 \text{ kipft}) + ((-0.007 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

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

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:

$L_{e,z} = 1.06 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.6262 \text{ ft}), (1.06 \text{ ft})]$$

$$L_{e,req} = 6.626 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.626 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.94657$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.864 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.68812 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.68812 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.34406$$

Status: **PASS**
Ratio: **0.340**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.129 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9563 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9563 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.9563 \text{ kipft/ft})) + (4 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7654 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.9563 \text{ kipft/ft})) + (3 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.9563 \text{ kipft/ft})) + (2 \times (-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.28288 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.9563 \text{ kipft/ft})) + ((-0.129 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.96359 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7654 \text{ ft})}{2}$$

$$p_a = 0.3574 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28288 \text{ kip/ft}^2)}{(0.3574 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79149$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.96359 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91771$$

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

Considering z-direction:

$H_o = -0.0023333 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.011333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.0023333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.011333 \text{ kipft/ft})) + (4 \times (-0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9525 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.011333 \text{ kipft/ft})) + (3 \times (-0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.011333 \text{ kipft/ft})) + (2 \times (-0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.00024241 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.011333 \text{ kipft/ft})) + ((-0.0023333 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.0012182 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9525 \text{ ft})}{2}$$

$$p_a = 0.37144 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00024241 \text{ kip/ft}^2)}{(0.37144 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00065263$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

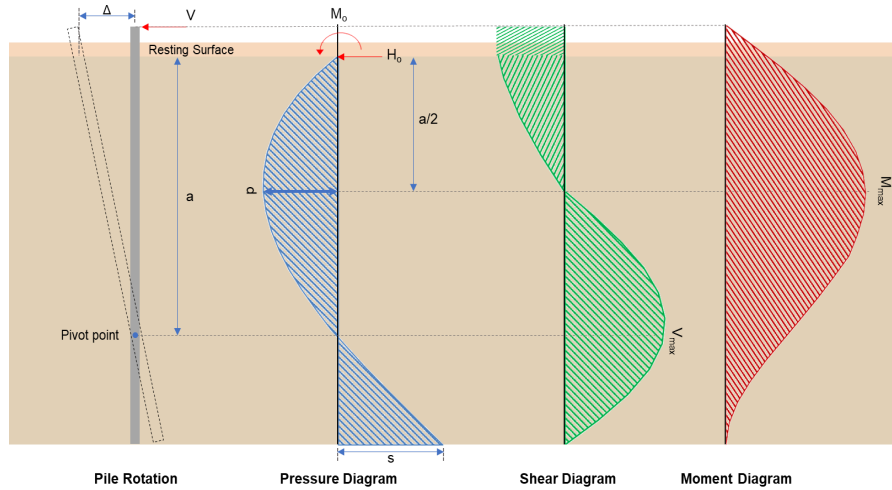
Status: **PASS**
Ratio: **0.000**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0012182 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0011602$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.646 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.21533 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.16 \text{ kipft}) + ((-0.646 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.0533 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0533 \text{ kipft/ft})}{(-0.21533 \text{ kip/ft})}$$

$$E = 23.467 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0533 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.21533 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.0533 \text{ kipft/ft})) + (4 \times (-0.21533 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7634 \text{ ft}$$

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

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

$$V_{max} = ((-0.21533 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2629 \text{ kip}$$

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

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

$$M_{max} = ((-0.21533 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.467 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.467 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.506 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.01 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0033333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.051 \text{ kipft}) + ((-0.01 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.017 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017 \text{ kipft/ft})}{(-0.0033333 \text{ kip/ft})}$$

$$E = 5.1 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.0033333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.017 \text{ kipft/ft})) + (4 \times (-0.0033333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9454 \text{ ft}$$

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

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

$$V_{max} = ((-0.0033333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9454 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9454 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019519 \text{ kip}$$

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

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

$$M_{max} = ((-0.0033333 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5.1 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9454 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.062041 \text{ kipft}$$

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,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(6.876 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.963 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-44.963 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$Ratio = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

Since longitudinal reinforcement is \leq No. 10 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.876 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0046071$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

22.5.2.2

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

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

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

$$\lambda_s = 0.71796$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.876 \text{ kip} \rightarrow 6876 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6876 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.71 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(203.86 \text{ kip}), (82.71 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.71 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.71 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.572 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.2629 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.2629 \text{ kip})}{(78.572 \text{ kip})}$ $Ratio = 0.054254$ <p>Considering z-direction:</p> <p>$V_{max} = 0.019519 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.019519 \text{ kip})}{(78.572 \text{ kip})}$ $Ratio = 0.00024843$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 14.506 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.506 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.2135$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.062041 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.062041 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.00091308$	<p>Status: PASS Ratio: 0.000</p>