

Your Project Calculations



Project Name: USDCampusPhase1-B2

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=USDCampusPhase1-B2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=BNEoBcUE5VFpVEReAKVi6avYah1irrXnjEgHHVf2vtKqYLy3KoZXb6rUy7OuSTD

Array Specification

Product:	Beam
Unique ID:	5P-19.75-8TOP-SD-57-L-5Hx14W-FC9I
Duty Classification:	SD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	14
Total Number of Modules:	70
Desired Tilt Angle:	10
Front Edge Clearance:	15
Total Array Height at Tilt:	18.25 ft
Total Frame Length:	96.00 ft
Frame Weight:	4869 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	97.53 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft
Rail Check:	PASS (46% utilized)

Support Specifications

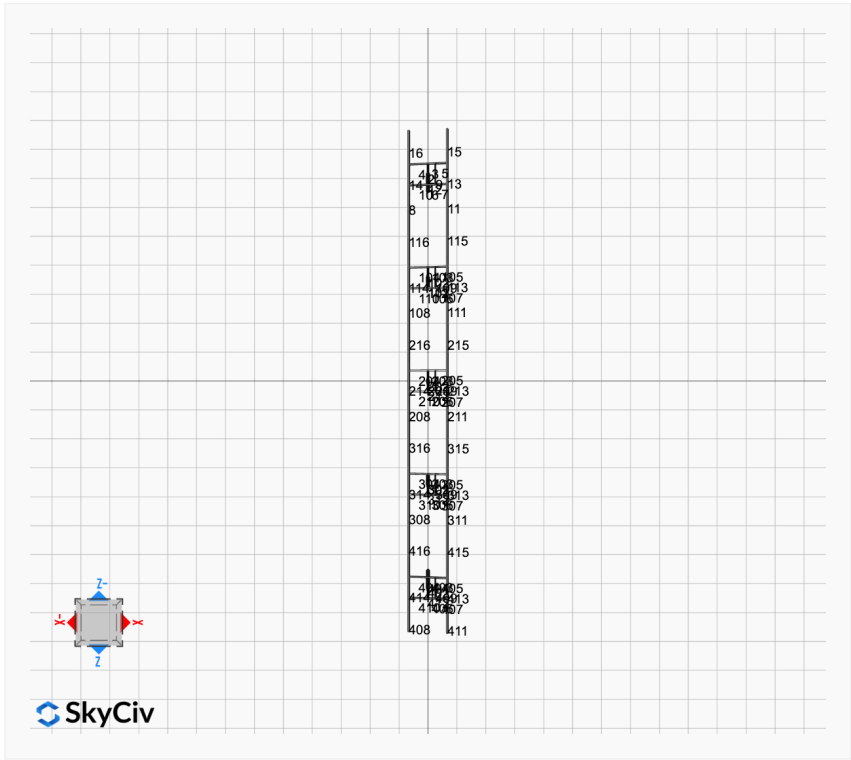
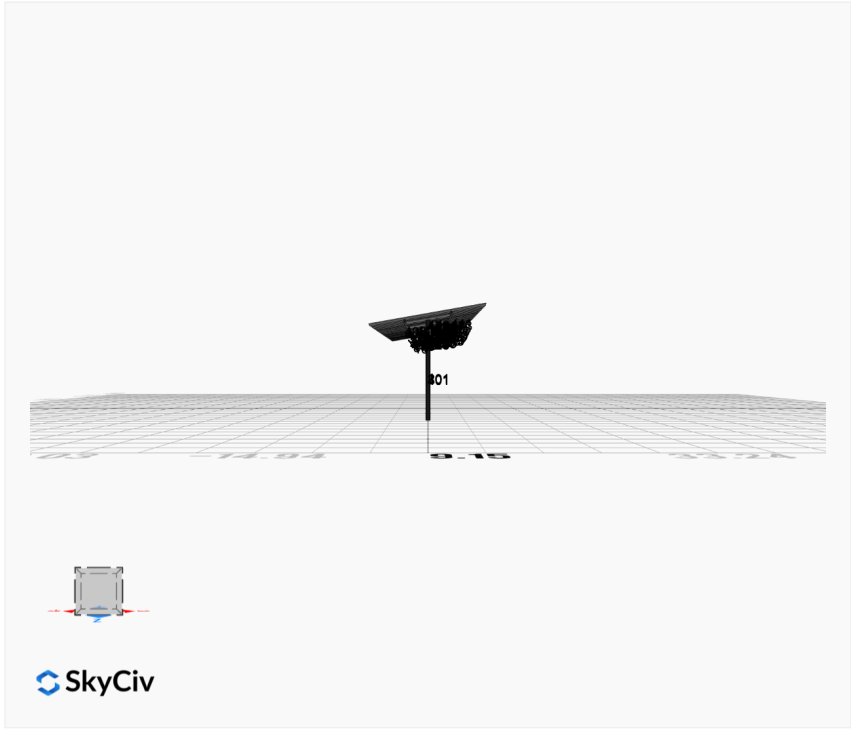
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	16.63 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

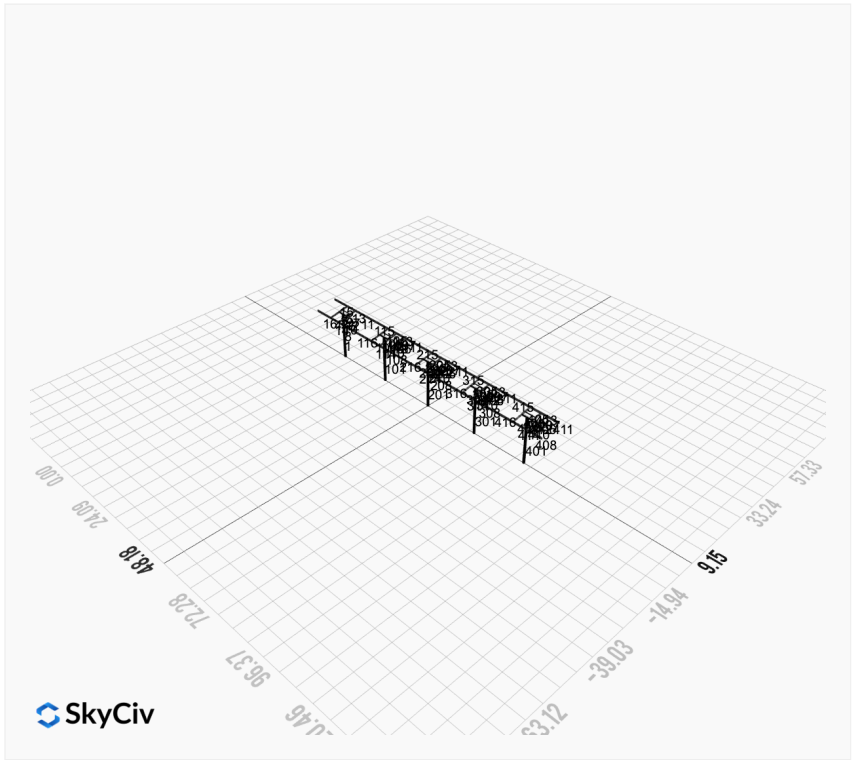
Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 7.00 ft Pile 2: 7.00 ft Pile 3: 7.00 ft Pile 4: 7.00 ft Pile 5: 7.00 ft
Foundation Volume:	9.163 y ³
Foundation Result:	PASSED
Mount Twist:	0.019318 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





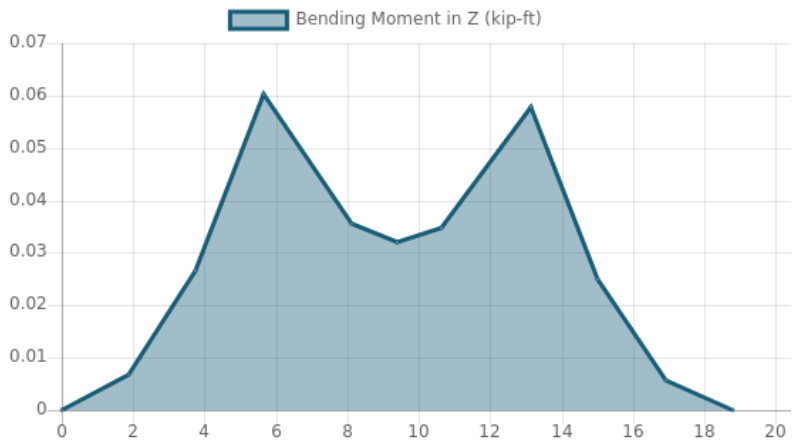
Rail Design Check

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

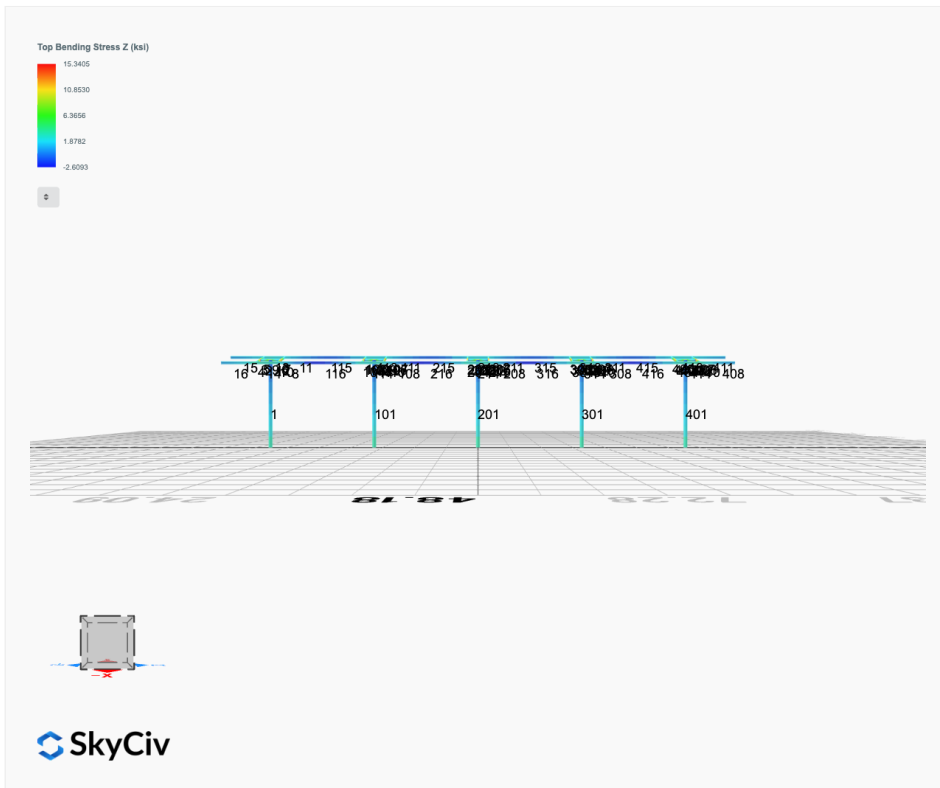
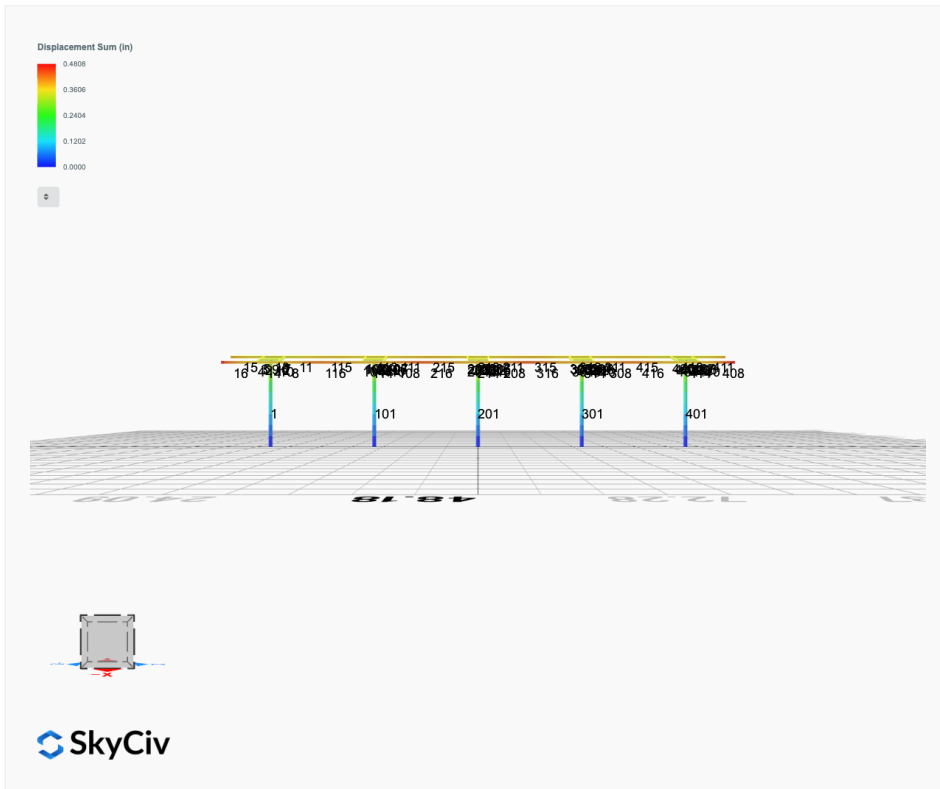


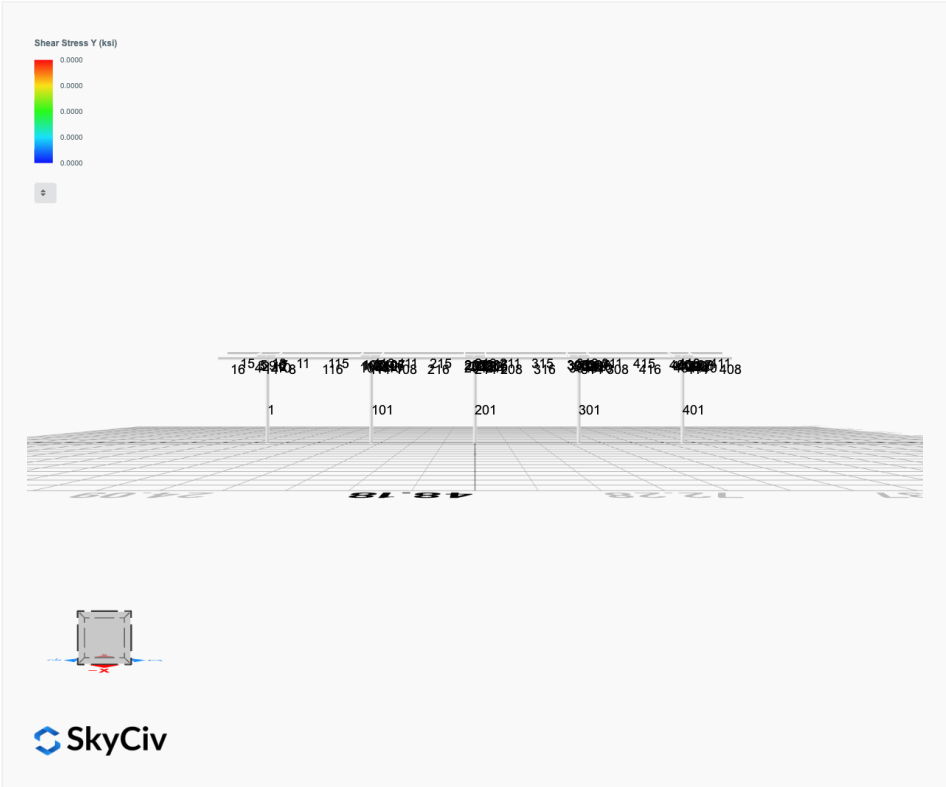
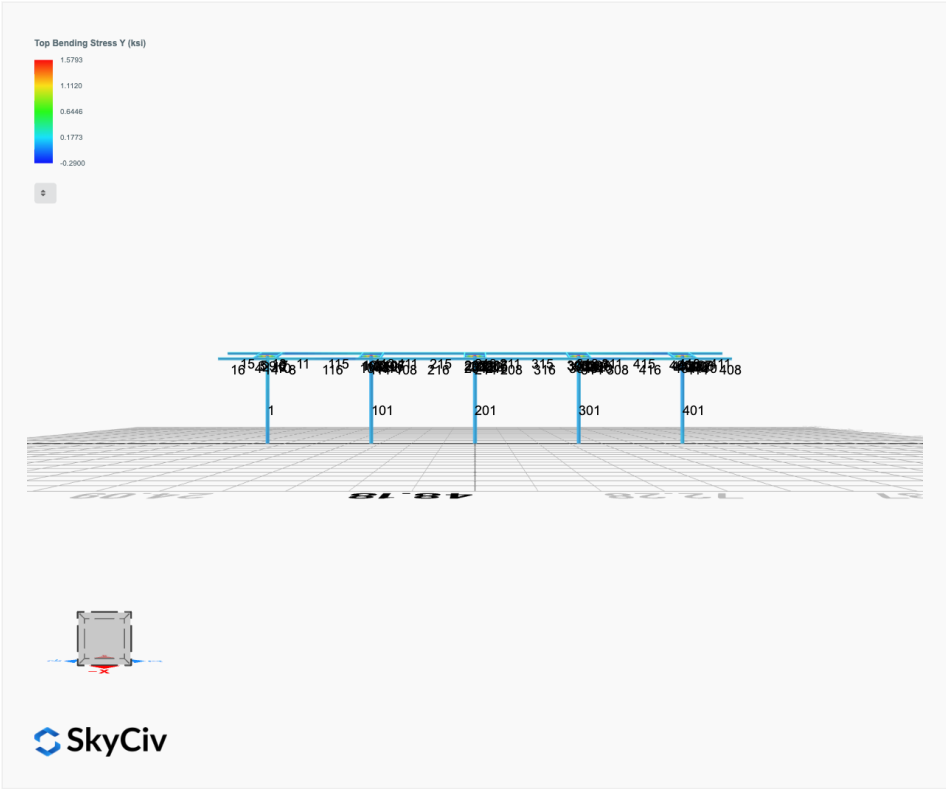
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	15.71619624	0.456	PASS
Material Yield	34.5	15.71619624	0.456	PASS
Material Strength	37	15.71619624	0.425	PASS

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	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 2. D + L	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 3. D + (S or Lr or R)	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 3. D + (S or Lr or R)	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 5b. D + 0.7E	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0012	2.5318	-0.0064	-0.0329	0.0116	0.0404
ULS: 8. 0.6D + 0.7E	-0.0007	1.5191	-0.0038	-0.0198	0.0070	0.0243
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3794	4.6462	-0.0126	-0.0658	0.0124	8.1087
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3794	4.6462	-0.0126	-0.0658	0.0124	8.1087
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2618	1.0492	-0.0016	-0.0078	0.0085	-3.0400
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2327	1.2410	-0.0032	-0.0158	0.0146	-8.6334
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2848	4.1176	-0.0111	-0.0576	0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2848	4.1176	-0.0111	-0.0576	0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1960	1.4198	-0.0028	-0.0141	0.0093	-2.2699
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1742	1.5637	-0.0040	-0.0201	0.0138	-6.4650
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2848	4.1176	-0.0111	-0.0576	0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2848	4.1176	-0.0111	-0.0576	0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1960	1.4198	-0.0028	-0.0141	0.0093	-2.2699
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1742	1.5637	-0.0040	-0.0201	0.0138	-6.4650
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3789	3.6334	-0.0101	-0.0526	0.0077	8.0926
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3789	3.6334	-0.0101	-0.0526	0.0077	8.0926
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2623	0.0365	0.0010	0.0053	0.0039	-3.0562
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2332	0.2283	-0.0006	-0.0026	0.0099	-8.6496

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	6.5621
Shear X	-0.6321
Shear Z	-0.0180
Moment X	-0.0944
Moment Y (Twist)	0.0193
Moment Z	14.7748

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	4.6462
Shear X	-0.3794
Shear Z	-0.0126
Moment X	-0.0658
Moment Y (Twist)	0.0146
Moment Z	8.6496

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	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 2. D + L	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 3. D + (S or Lr or R)	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 3. D + (S or Lr or R)	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 5b. D + 0.7E	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0014	2.6532	0.0023	0.0124	-0.0023	0.0027
ULS: 8. 0.6D + 0.7E	0.0008	1.5919	0.0014	0.0074	-0.0014	0.0016
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3910	4.9001	0.0058	0.0303	-0.0118	8.4039
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3910	4.9001	0.0058	0.0303	-0.0118	8.4039
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2781	1.0773	0.0003	0.0017	0.0013	-3.2244
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2388	1.2821	-0.0002	-0.0007	0.0075	-8.9822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2929	4.3384	0.0049	0.0258	-0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2929	4.3384	0.0049	0.0258	-0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2089	1.4713	0.0008	0.0044	0.0004	-2.4176
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1794	1.6249	0.0004	0.0026	0.0050	-6.7360
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2929	4.3384	0.0049	0.0258	-0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2929	4.3384	0.0049	0.0258	-0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2089	1.4713	0.0008	0.0044	0.0004	-2.4176
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1794	1.6249	0.0004	0.0026	0.0050	-6.7360
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3916	3.8388	0.0049	0.0253	-0.0108	8.4029
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3916	3.8388	0.0049	0.0253	-0.0108	8.4029
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2775	0.0160	-0.0007	-0.0033	0.0022	-3.2254
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2382	0.2208	-0.0012	-0.0056	0.0084	-8.9833

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	6.9286
Shear X	-0.6539
Shear Z	0.0087
Moment X	0.0451
Moment Y (Twist)	0.0190
Moment Z	15.3586

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	4.9001
Shear X	-0.3916
Shear Z	0.0058
Moment X	0.0303
Moment Y (Twist)	0.0118
Moment Z	8.9833

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	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 2. D + L	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 3. D + (S or Lr or R)	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 3. D + (S or Lr or R)	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 5b. D + 0.7E	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0003	2.6697	-0.0000	0.0000	0.0000	0.0256
ULS: 8. 0.6D + 0.7E	-0.0002	1.6018	-0.0000	0.0000	0.0000	0.0153
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3970	4.9370	-0.0000	0.0000	0.0000	8.5261
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3970	4.9370	-0.0000	0.0000	0.0000	8.5261
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2793	1.0797	-0.0000	0.0000	0.0000	-3.2352
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2398	1.2855	-0.0000	0.0000	0.0000	-9.0664
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2978	4.3702	-0.0000	0.0000	0.0000	6.4009
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2978	4.3702	-0.0000	0.0000	0.0000	6.4009
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2094	1.4772	-0.0000	0.0000	0.0000	-2.4200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1798	1.6316	-0.0000	0.0000	0.0000	-6.7934

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	-0.2978	4.3702	-0.0000	0.0000	0.0000	6.4009
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2978	4.3702	-0.0000	0.0000	0.0000	6.4009
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2094	1.4772	-0.0000	0.0000	0.0000	-2.4200
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1798	1.6316	-0.0000	0.0000	0.0000	-6.7934
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3968	3.8691	-0.0000	0.0000	0.0000	8.5159
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3968	3.8691	-0.0000	0.0000	0.0000	8.5159
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2794	0.0119	-0.0000	0.0000	0.0000	-3.2454
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2399	0.2177	-0.0000	0.0000	0.0000	-9.0766

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	6.9825
Shear X	-0.6612
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	15.5170

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	4.9370
Shear X	-0.3970
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	9.0766

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	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 2. D + L	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 3. D + (S or Lr or R)	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 3. D + (S or Lr or R)	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 5b. D + 0.7E	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0014	2.6532	-0.0023	-0.0124	0.0023	0.0027
ULS: 8. 0.6D + 0.7E	0.0008	1.5919	-0.0014	-0.0074	0.0014	0.0016
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3910	4.9001	-0.0058	-0.0303	0.0118	8.4039
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3910	4.9001	-0.0058	-0.0303	0.0118	8.4039
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2781	1.0773	-0.0003	-0.0017	-0.0013	-3.2244
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2388	1.2821	0.0002	0.0007	-0.0075	-8.9822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2929	4.3384	-0.0049	-0.0258	0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2929	4.3384	-0.0049	-0.0258	0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2089	1.4713	-0.0008	-0.0044	-0.0004	-2.4176
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1794	1.6249	-0.0004	-0.0026	-0.0050	-6.7360
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2929	4.3384	-0.0049	-0.0258	0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2929	4.3384	-0.0049	-0.0258	0.0094	6.3036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2089	1.4713	-0.0008	-0.0044	-0.0004	-2.4176
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1794	1.6249	-0.0004	-0.0026	-0.0050	-6.7360
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3916	3.8388	-0.0049	-0.0253	0.0108	8.4029
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3916	3.8388	-0.0049	-0.0253	0.0108	8.4029
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2775	0.0160	0.0007	0.0033	-0.0022	-3.2254
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2382	0.2208	0.0012	0.0056	-0.0084	-8.9833

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	6.9286
Shear X	-0.6539
Shear Z	-0.0087
Moment X	-0.0451
Moment Y (Twist)	0.0190
Moment Z	15.3586

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	4.9001
Shear X	-0.3916
Shear Z	-0.0058
Moment X	-0.0303
Moment Y (Twist)	0.0118
Moment Z	8.9833

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 2. D + L	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 3. D + (S or Lr or R)	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 3. D + (S or Lr or R)	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 5b. D + 0.7E	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0012	2.5318	0.0064	0.0329	-0.0116	0.0404
ULS: 8. 0.6D + 0.7E	-0.0007	1.5191	0.0038	0.0198	-0.0070	0.0243
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3794	4.6462	0.0126	0.0658	-0.0124	8.1087
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3794	4.6462	0.0126	0.0658	-0.0124	8.1087
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.2618	1.0492	0.0016	0.0078	-0.0085	-3.0400
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2327	1.2410	0.0032	0.0158	-0.0146	-8.6335
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2848	4.1176	0.0111	0.0576	-0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2848	4.1176	0.0111	0.0576	-0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1960	1.4198	0.0028	0.0141	-0.0093	-2.2699
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1742	1.5637	0.0040	0.0201	-0.0138	-6.4650
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2848	4.1176	0.0111	0.0576	-0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2848	4.1176	0.0111	0.0576	-0.0122	6.0917
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1960	1.4198	0.0028	0.0141	-0.0093	-2.2699
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1742	1.5637	0.0040	0.0201	-0.0138	-6.4650
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3789	3.6334	0.0101	0.0526	-0.0077	8.0926
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3789	3.6334	0.0101	0.0526	-0.0077	8.0926
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.2623	0.0365	-0.0010	-0.0053	-0.0039	-3.0562
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2332	0.2283	0.0006	0.0026	-0.0099	-8.6496

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	6.5621
Shear X	-0.6321
Shear Z	0.0180
Moment X	0.0944
Moment Y (Twist)	0.0193
Moment Z	14.7749

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	4.6462
Shear X	-0.3794
Shear Z	0.0126
Moment X	0.0658
Moment Y (Twist)	0.0146
Moment Z	8.6496

Project Details

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

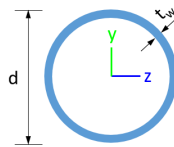


Design Input Information

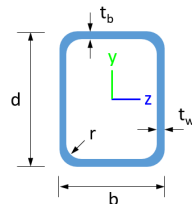
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

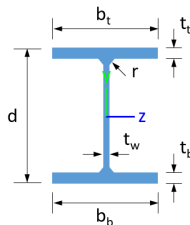
Section Dimensions



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

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	18.13	6.45	30.09	45.74
215	120.60	68.63	15.30	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	117.88	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	117.88	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	17.96	6.45	30.09	45.74
314	120.60	98.23	18.31	6.45	30.09	45.74
315	120.60	68.63	15.57	6.45	30.09	45.74
316	120.60	68.63	15.71	6.45	30.09	45.74
401	377.97	93.23	83.29	83.29	113.39	113.39
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.02	10.99	4.60	29.14	16.61
404	79.65	72.01	10.99	4.60	29.14	16.61
405	79.65	73.44	10.99	4.60	29.14	16.61
406	79.65	74.02	10.99	4.60	29.14	16.61
407	79.65	73.44	10.99	4.60	29.14	16.61
408	120.60	34.69	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.01	10.99	4.60	29.14	16.61
411	120.60	34.69	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	98.23	19.19	6.45	30.09	45.74
414	120.60	98.23	19.37	6.45	30.09	45.74
415	120.60	68.63	14.89	6.45	30.09	45.74
416	120.60	68.63	15.98	6.45	30.09	45.74

Design Ratio

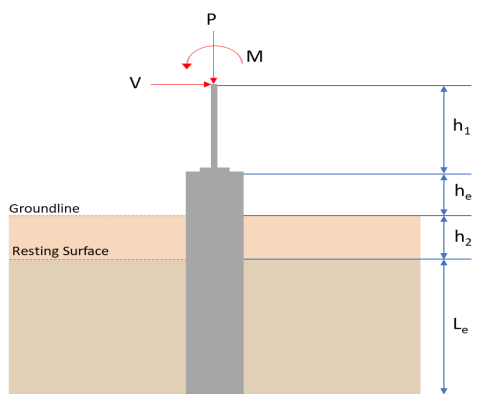
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.070	0.177	0.002	0.006	0.000	0.202	#13	0.713	Not Required	Pass
2	0.000	0.342	0.044	0.071	0.007	0.386	#13	0.034	Not Required	Pass
3	0.002	0.569	0.016	0.058	0.002	0.586	#13	0.044	Not Required	Pass
4	0.002	0.449	0.038	0.046	0.005	0.469	#13	0.078	Not Required	Pass
5	0.002	0.352	0.041	0.057	0.006	0.360	#13	0.073	Not Required	Pass
6	0.002	0.559	0.015	0.057	0.001	0.571	#13	0.044	Not Required	Pass
7	0.002	0.346	0.042	0.056	0.006	0.354	#13	0.073	Not Required	Pass
8	0.000	0.034	0.017	0.029	0.002	0.042	#13	0.059	Not Required	Pass

9	0.002	0.064	0.013	0.001	0.000	0.077	#13	0.198	Not Required	Pass
10	0.002	0.436	0.041	0.044	0.005	0.460	#13	0.078	Not Required	Pass
11	0.000	0.041	0.018	0.037	0.002	0.049	#13	0.088	Not Required	Pass
12	0.000	0.330	0.043	0.069	0.007	0.373	#13	0.034	Not Required	Pass
13	0.001	0.198	0.041	0.047	0.002	0.228	#13	0.265	Not Required	Pass
14	0.001	0.162	0.041	0.037	0.002	0.187	#13	0.177	Not Required	Pass
15	0.000	0.085	0.022	0.028	0.001	0.104	#13	Not Required	Not Required	Pass
16	0.000	0.067	0.022	0.022	0.001	0.086	#13	Not Required	Not Required	Pass
101	0.074	0.184	0.001	0.006	0.000	0.210	#13	0.713	Not Required	Pass
102	0.000	0.350	0.044	0.074	0.007	0.394	#13	0.034	Not Required	Pass
103	0.002	0.594	0.017	0.060	0.002	0.611	#13	0.044	Not Required	Pass
104	0.002	0.469	0.038	0.048	0.005	0.496	#13	0.078	Not Required	Pass
105	0.002	0.368	0.038	0.060	0.006	0.377	#13	0.073	Not Required	Pass
106	0.002	0.602	0.018	0.061	0.002	0.617	#13	0.044	Not Required	Pass
107	0.002	0.373	0.039	0.060	0.006	0.382	#13	0.073	Not Required	Pass
108	0.000	0.039	0.014	0.028	0.002	0.045	#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	0.473	0.037	0.048	0.005	0.494	#13	0.078	Not Required	Pass
111	0.000	0.048	0.014	0.036	0.002	0.055	#13	0.088	Not Required	Pass
112	0.000	0.356	0.045	0.075	0.008	0.401	#13	0.034	Not Required	Pass
113	0.001	0.165	0.038	0.046	0.002	0.192	#13	0.265	Not Required	Pass
114	0.001	0.137	0.038	0.036	0.002	0.160	#13	0.177	Not Required	Pass
115	0.000	0.159	0.022	0.035	0.002	0.178	#13	0.439	Not Required	Pass
116	0.000	0.126	0.022	0.028	0.002	0.146	#13	0.293	Not Required	Pass
201	0.075	0.186	0.000	0.006	0.000	0.212	#13	0.713	Not Required	Pass
202	0.000	0.356	0.045	0.075	0.008	0.401	#13	0.034	Not Required	Pass
203	0.002	0.604	0.018	0.061	0.002	0.619	#13	0.044	Not Required	Pass
204	0.002	0.475	0.038	0.048	0.005	0.499	#13	0.078	Not Required	Pass
205	0.002	0.374	0.039	0.061	0.006	0.382	#13	0.073	Not Required	Pass
206	0.002	0.604	0.018	0.061	0.002	0.619	#13	0.044	Not Required	Pass
207	0.002	0.374	0.039	0.061	0.006	0.383	#13	0.073	Not Required	Pass
208	0.000	0.037	0.014	0.028	0.002	0.042	#13	0.088	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	0.475	0.038	0.048	0.005	0.499	#13	0.078	Not Required	Pass
211	0.000	0.047	0.015	0.036	0.002	0.051	#13	0.088	Not Required	Pass
212	0.000	0.356	0.045	0.075	0.008	0.401	#13	0.034	Not Required	Pass
213	0.001	0.169	0.039	0.046	0.002	0.195	#13	0.265	Not Required	Pass
214	0.001	0.140	0.038	0.037	0.002	0.160	#13	0.265	Not Required	Pass
215	0.000	0.163	0.022	0.036	0.002	0.182	#13	0.439	Not Required	Pass
216	0.000	0.129	0.022	0.028	0.002	0.148	#13	0.439	Not Required	Pass
301	0.074	0.184	0.001	0.006	0.000	0.210	#13	0.713	Not Required	Pass
302	0.000	0.356	0.045	0.075	0.008	0.401	#13	0.034	Not Required	Pass
303	0.002	0.602	0.018	0.061	0.002	0.617	#13	0.044	Not Required	Pass
304	0.002	0.473	0.037	0.048	0.005	0.494	#13	0.078	Not Required	Pass
305	0.002	0.373	0.039	0.060	0.006	0.382	#13	0.073	Not Required	Pass
306	0.002	0.594	0.017	0.060	0.002	0.611	#13	0.044	Not Required	Pass
307	0.002	0.368	0.038	0.060	0.006	0.377	#13	0.073	Not Required	Pass
308	0.000	0.038	0.016	0.028	0.002	0.040	#13	0.059	Not Required	Pass
309	0.001	0.061	0.012	0.001	0.000	0.073	#13	0.198	Not Required	Pass
310	0.002	0.469	0.038	0.048	0.005	0.496	#13	0.078	Not Required	Pass
311	0.000	0.048	0.016	0.035	0.002	0.050	#13	0.088	Not Required	Pass
312	0.000	0.350	0.044	0.074	0.007	0.394	#13	0.034	Not Required	Pass
313	0.001	0.165	0.038	0.046	0.002	0.192	#13	0.265	Not Required	Pass
314	0.001	0.137	0.038	0.036	0.002	0.160	#13	0.265	Not Required	Pass

315	0.000	0.164	0.022	0.036	0.002	0.183	#13	0.439	Not Required	Pass
316	0.000	0.130	0.022	0.028	0.002	0.149	#13	0.439	Not Required	Pass
401	0.070	0.177	0.002	0.006	0.000	0.202	#13	0.713	Not Required	Pass
402	0.000	0.330	0.043	0.069	0.007	0.373	#13	0.034	Not Required	Pass
403	0.002	0.559	0.015	0.057	0.001	0.571	#13	0.044	Not Required	Pass
404	0.002	0.436	0.041	0.044	0.005	0.460	#13	0.078	Not Required	Pass
405	0.002	0.346	0.042	0.056	0.006	0.354	#13	0.073	Not Required	Pass
406	0.002	0.569	0.016	0.058	0.002	0.586	#13	0.044	Not Required	Pass
407	0.002	0.352	0.041	0.057	0.006	0.360	#13	0.073	Not Required	Pass
408	0.000	0.067	0.022	0.022	0.001	0.086	#13	Not Required	Not Required	Pass
409	0.002	0.064	0.013	0.001	0.000	0.077	#13	0.198	Not Required	Pass
410	0.002	0.449	0.038	0.046	0.005	0.469	#13	0.078	Not Required	Pass
411	0.000	0.085	0.022	0.028	0.001	0.104	#13	Not Required	Not Required	Pass
412	0.000	0.342	0.044	0.071	0.007	0.386	#13	0.034	Not Required	Pass
413	0.001	0.198	0.041	0.047	0.002	0.228	#13	0.177	Not Required	Pass
414	0.001	0.162	0.041	0.037	0.002	0.187	#13	0.177	Not Required	Pass
415	0.000	0.157	0.022	0.037	0.002	0.176	#13	0.439	Not Required	Pass
416	0.000	0.125	0.022	0.029	0.002	0.143	#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 $L = 7$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="414 1075 1189 1176"> <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="670 1265 933 1433"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.646</td> <td>6.562</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.379</td> <td>-0.632</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>-0.094</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.650</td> <td>14.775</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)	4.646	6.562	V_x (kip)	-0.379	-0.632	V_z (kip)	-0.013	-0.018	M_x (kipft)	-0.066	-0.094	M_z (kipft)	8.650	14.775	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.379 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.12633 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.65 \text{ kipft}) + ((-0.379 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.8833 \text{ kipft/ft}$$

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:

$$L_{e,x} = 6.5736 \text{ ft} - \text{Required depth in x-direction,}$$

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:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.5736 \text{ ft}), (1.3067 \text{ ft})]$$

$$L_{e,req} = 6.574 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.574 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.93914$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.646 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.65727 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.65727 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32864$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.12633 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.8833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.8833 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.8833 \text{ kipft/ft})) + (4 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7657 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.8833 \text{ kipft/ft})) + (3 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.8833 \text{ kipft/ft})) + (2 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.27551 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.8833 \text{ kipft/ft})) + ((-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.9391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7657 \text{ ft})}{2}$$

$$p_a = 0.35743 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27551 \text{ kip/ft}^2)}{(0.35743 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77081$$

Status: **PASS**
Ratio: **0.770**

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

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

$$\text{Ratio} = \frac{(0.9391 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89438$$

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

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,

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

$$a = \frac{(4 \times (0.022 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.022 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9461 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.022 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.022 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.000040569 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.022 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.0026287 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9461 \text{ ft})}{2}$$

$$p_a = 0.37095 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.000040569 \text{ kip/ft}^2)}{(0.37095 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00010936$$

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

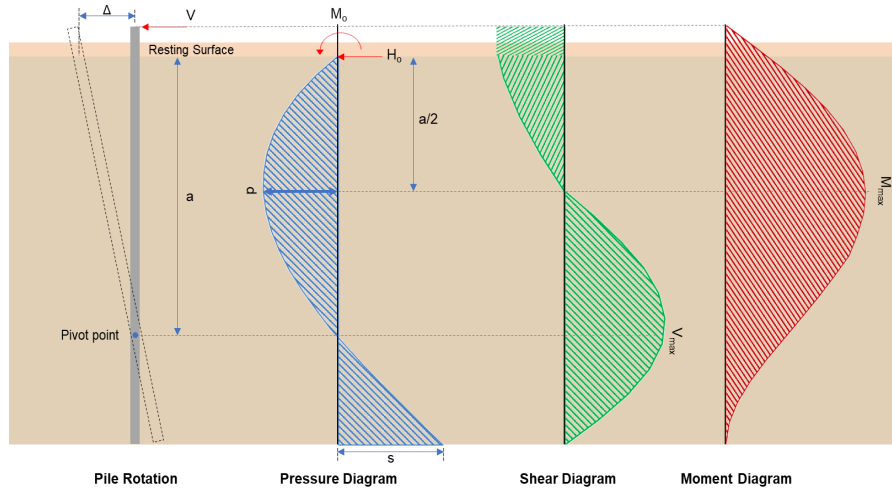
Status: **PASS**
Ratio: **0.000**

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

$$Ratio = \frac{(0.0026287 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0025036$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.632 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.21067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.775 \text{ kipft}) + ((-0.632 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.925 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.925 \text{ kipft/ft})}{(-0.21067 \text{ kip/ft})}$$

$$E = 23.378 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.925 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.21067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.925 \text{ kipft/ft})) + (4 \times (-0.21067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7637 \text{ ft}$$

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

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

$$V_{max} = ((-0.21067 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7637 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7637 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.1562 \text{ kip}$$

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

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

$$M_{max} = ((-0.21067 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.378 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.142 \text{ kipft}$$

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,

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

$$M_o = \frac{(0.094 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.031333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.031333 \text{ kipft/ft})}{(-0.006 \text{ kip/ft})}$$

$$E = 5.2222 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.031333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.006 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.031333 \text{ kipft/ft})) + (4 \times (-0.006 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9419 \text{ ft}$$

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

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

$$V_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9419 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9419 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.035688 \text{ kip}$$

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

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

$$M_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5.2222 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1136 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.562 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.973 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.973 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(6.562 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0043967$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.562 \text{ kip} \rightarrow 6562 \text{ lbf}$.

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6562 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.657 \text{ kip}$$

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)

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

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

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

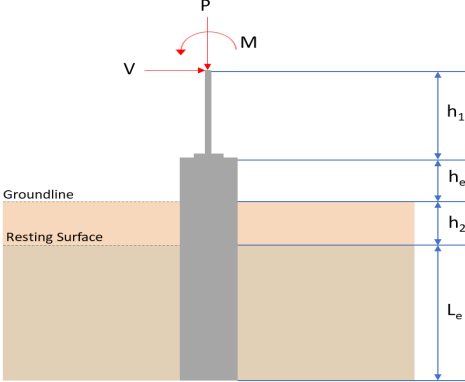
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(203.86 \text{ kip}), (82.657 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.657 \text{ kip}$</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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.657 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.538 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.1562 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.1562 \text{ kip})}{(78.538 \text{ kip})}$ $Ratio = 0.05292$ <p>Considering z-direction:</p> <p>$V_{max} = 0.035688 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.035688 \text{ kip})}{(78.538 \text{ kip})}$ $Ratio = 0.0004544$	<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: $M_{max} = 14.142 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.142 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.20813$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.1136 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.1136 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.0016719$	<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>4.900</td> <td>6.929</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.392</td> <td>-0.654</td> </tr> <tr> <td>V_z (kip)</td> <td>0.006</td> <td>0.009</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.030</td> <td>0.045</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.983</td> <td>15.359</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)	4.900	6.929	V_x (kip)	-0.392	-0.654	V_z (kip)	0.006	0.009	M_x (kipft)	0.030	0.045	M_z (kipft)	8.983	15.359	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.392 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.13067 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.983 \text{ kipft}) + ((-0.392 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.9943 \text{ kipft/ft}$$

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:

$$L_{e,x} = 6.652 \text{ ft} - \text{Required depth in x-direction,}$$

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.006 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.002 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.03 \text{ kipft}) + ((0.006 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.01 \text{ kipft/ft}$$

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:

$$L_{e,z} = 1.1373 \text{ ft} - \text{Required depth in z-direction,}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.652 \text{ ft}), (1.1373 \text{ ft})]$$

$$L_{e,req} = 6.652 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.652 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.95029$$

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

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

$$q = \frac{(4.9 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.69321 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.69321 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.3466$$

Status: **PASS**
Ratio: **0.350**

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:

$H_o = -0.13067 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9943 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9943 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.9943 \text{ kipft/ft})) + (4 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

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

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

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

$$p = \frac{1.178 \times [(4 \times (2.9943 \text{ kipft/ft})) + (3 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.9943 \text{ kipft/ft})) + (2 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.28651 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.9943 \text{ kipft/ft})) + ((-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.97597 \text{ kip/ft}^2$$

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

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

$$\text{Ratio} = \frac{(0.28651 \text{ kip/ft}^2)}{(0.3574 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.80164$$

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

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

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

$$\text{Ratio} = \frac{(0.97597 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9295$$

Status: **PASS**
Ratio: **0.930**

Considering z-direction:

$H_o = 0.002 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.01 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.01 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.002 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.01 \text{ kipft/ft})) + (4 \times (0.002 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9483 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.01 \text{ kipft/ft})) + (3 \times (0.002 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.01 \text{ kipft/ft})) + (2 \times (0.002 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.0027871 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.01 \text{ kipft/ft})) + ((0.002 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.0065398 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9483 \text{ ft})}{2}$$

$$p_a = 0.37112 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0027871 \text{ kip/ft}^2)}{(0.37112 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0075099$$

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

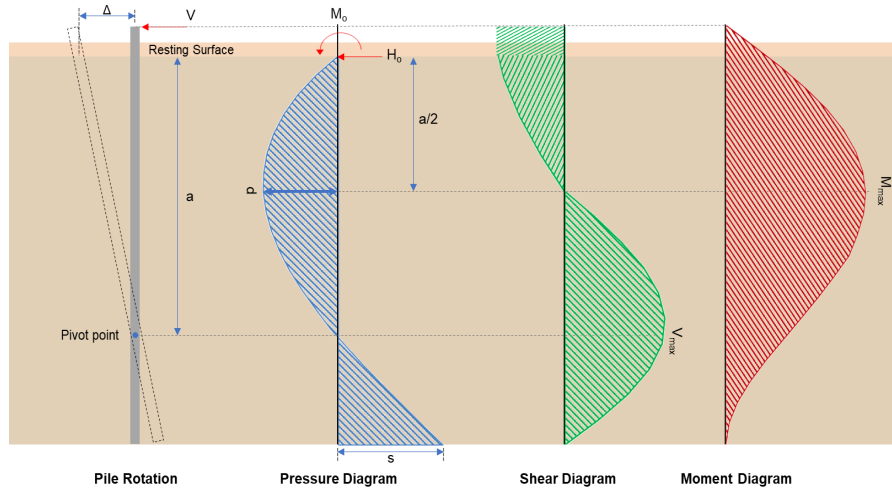
Status: **PASS**
Ratio: **0.010**

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

$$Ratio = \frac{(0.0065398 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0062284$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.654 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.218 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.359 \text{ kipft}) + ((-0.654 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1197 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1197 \text{ kipft/ft})}{(-0.218 \text{ kip/ft})}$$

$$E = 23.485 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1197 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.218 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.1197 \text{ kipft/ft})) + (4 \times (-0.218 \text{ kip/ft}) \times (7 \text{ ft}))}$$

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

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

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

$$V_{max} = ((-0.218 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.3185 \text{ kip}$$

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

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

$$M_{max} = ((-0.218 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.485 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.696 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.009 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.003 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.045 \text{ kipft}) + ((0.009 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.015 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015 \text{ kipft/ft})}{(0.003 \text{ kip/ft})}$$

$$E = 5 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.003 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.015 \text{ kipft/ft})) + (4 \times (0.003 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9483 \text{ ft}$$

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

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

$$V_{max} = ((0.003 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9483 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9483 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.017341 \text{ kip}$$

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

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

$$M_{max} = ((0.003 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.055047 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.929 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.962 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.962 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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, LFRD)

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

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

$$\text{Ratio} = \frac{(6.929 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0046426$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.929 \text{ kip} \rightarrow 6929 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6929 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.719 \text{ kip}$$

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)

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

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

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

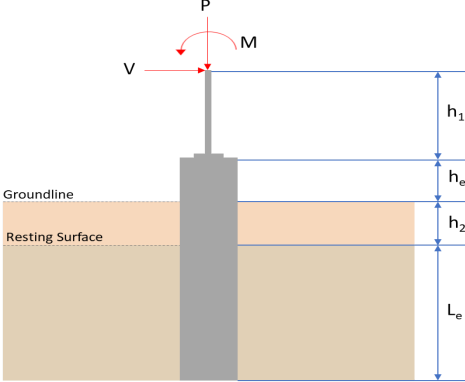
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.719 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.719 \text{ kip}$</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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.719 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.578 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.3185 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.3185 \text{ kip})}{(78.578 \text{ kip})}$ $Ratio = 0.054959$ <p>Considering z-direction:</p> <p>$V_{max} = 0.017341 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.017341 \text{ kip})}{(78.578 \text{ kip})}$ $Ratio = 0.00022069$	<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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <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: $M_{max} = 14.696 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.696 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.21629$	<p>Status: PASS Ratio: 0.220</p>
	<p>Considering z-direction: $M_{max} = 0.055047 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.055047 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.00081015$	<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>4.937</td> <td>6.982</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.397</td> <td>-0.661</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.077</td> <td>15.517</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)	4.937	6.982	V_x (kip)	-0.397	-0.661	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	9.077	15.517	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.397 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.13233 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{D}$																											

	$M_o = \frac{(9.077 \text{ kipft}) + ((-0.397 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 3.0257 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 6.6719 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.6719 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.672 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_c - h_2$ $L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7 \text{ ft}$ <p>Ratio - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.672 \text{ ft})}{(7 \text{ ft})}$ $\text{Ratio} = 0.95314$	<p>Status: PASS Ratio: 0.950</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$ $A = 7.0686 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_c}{A}$ $q = \frac{(4.937 \text{ kip})}{(7.0686 \text{ ft}^2)}$ $q = 0.69844 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p>Ratio - Capacity</p> $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.69844 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.34922$	<p>Status: PASS Ratio: 0.350</p>
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$	

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.13233$ kip/ft - Lateral force per length of pile,

$M_o = 3.0257$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0257 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.13233 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (3.0257 \text{ kipft/ft})) + (4 \times (-0.13233 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7655 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.0257 \text{ kipft/ft})) + (3 \times (-0.13233 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (3.0257 \text{ kipft/ft})) + (2 \times (-0.13233 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.28928 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.0257 \text{ kipft/ft})) + ((-0.13233 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.98578 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7655 \text{ ft})}{2}$$

$$p_a = 0.35742 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28928 \text{ kip/ft}^2)}{(0.35742 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.80938$$

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

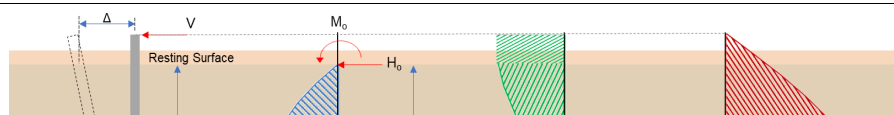
$$\text{Ratio} = \frac{s}{p_s}$$

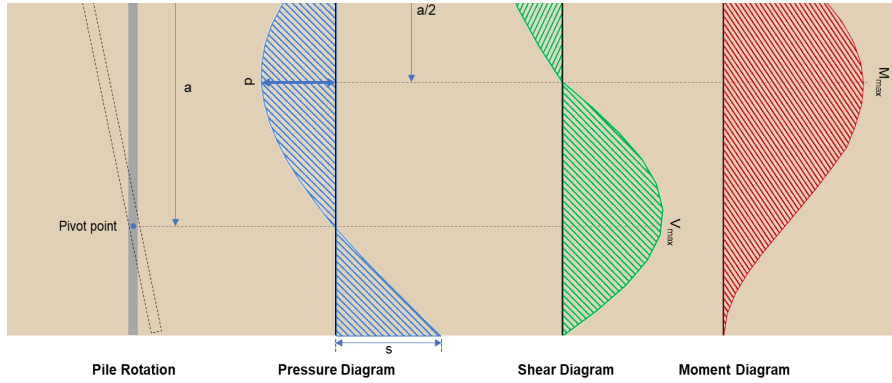
$$\text{Ratio} = \frac{(0.98578 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93884$$

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

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.661 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.22033 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.517 \text{ kipft}) + ((-0.661 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1723 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1723 \text{ kipft/ft})}{(-0.22033 \text{ kip/ft})}$$

$$E = 23.475 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1723 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.22033 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.1723 \text{ kipft/ft})) + (4 \times (-0.22033 \text{ kip/ft}) \times (7 \text{ ft}))}$$

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

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

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

$$V_{max} = ((-0.22033 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.475 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.475 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.3631 \text{ kip}$$

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

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

$$M_{max} = ((-0.22033 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.475 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.475 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.475 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.848 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.982 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.96 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.96 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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. 10Ø: 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

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

$$\text{Ratio} = \frac{(6.982 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0046781$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.982 \text{ kip} \rightarrow 6982 \text{ lbf}$.

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6982 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.728 \text{ kip}$$

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

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.728 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.728 \text{ kip}$</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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.728 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.584 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.3631 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.3631 \text{ kip})}{(78.584 \text{ kip})}$ $\text{Ratio} = 0.055522$	<p>Status: PASS Ratio: 0.060</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4580.4 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</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>	

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 14.848 \text{ kipft}$ - Maximum moment in the x-direction,

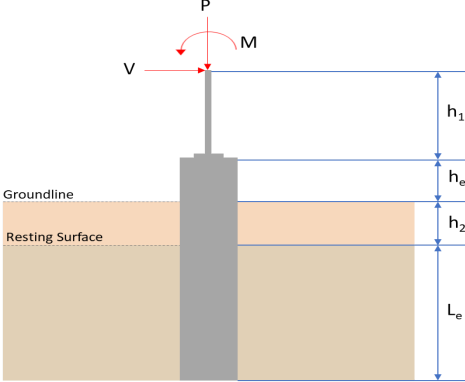
$Ratio$ - Capacity

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

$$Ratio = \frac{(14.848 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.21852$$

Status: **PASS**
Ratio: **0.220**

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 resting 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>4.900</td> <td>6.929</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.392</td> <td>-0.654</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.006</td> <td>-0.009</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.030</td> <td>-0.045</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.983</td> <td>15.359</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)	4.900	6.929	V_x (kip)	-0.392	-0.654	V_z (kip)	-0.006	-0.009	M_x (kipft)	-0.030	-0.045	M_z (kipft)	8.983	15.359	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.392 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.13067 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.983 \text{ kipft}) + ((-0.392 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.9943 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.652 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.006 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.002 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.03 \text{ kipft}) + ((-0.006 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.01 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.021 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.652 \text{ ft}), (1.021 \text{ ft})]$$

$$L_{e,req} = 6.652 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.652 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.95029$$

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

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

$$q = \frac{(4.9 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.69321 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.69321 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.3466$$

Status: **PASS**
Ratio: **0.350**

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:

$H_o = -0.13067 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9943 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9943 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.9943 \text{ kipft/ft})) + (4 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

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

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

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

$$p = \frac{1.178 \times [(4 \times (2.9943 \text{ kipft/ft})) + (3 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.9943 \text{ kipft/ft})) + (2 \times (-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.28651 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.9943 \text{ kipft/ft})) + ((-0.13067 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.97597 \text{ kip/ft}^2$$

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

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

$$\text{Ratio} = \frac{(0.28651 \text{ kip/ft}^2)}{(0.3574 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.80164$$

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

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

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

$$\text{Ratio} = \frac{(0.97597 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9295$$

Status: **PASS**
Ratio: **0.930**

Considering z-direction:

$H_o = -0.002 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.01 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.01 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.002 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.01 \text{ kipft/ft})) + (4 \times (-0.002 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9483 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.01 \text{ kipft/ft})) + (3 \times (-0.002 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.01 \text{ kipft/ft})) + (2 \times (-0.002 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.000048082 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.01 \text{ kipft/ft})) + ((-0.002 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.0011541 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9483 \text{ ft})}{2}$$

$$p_a = 0.37112 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.000048082 \text{ kip/ft}^2)}{(0.37112 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00012956$$

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

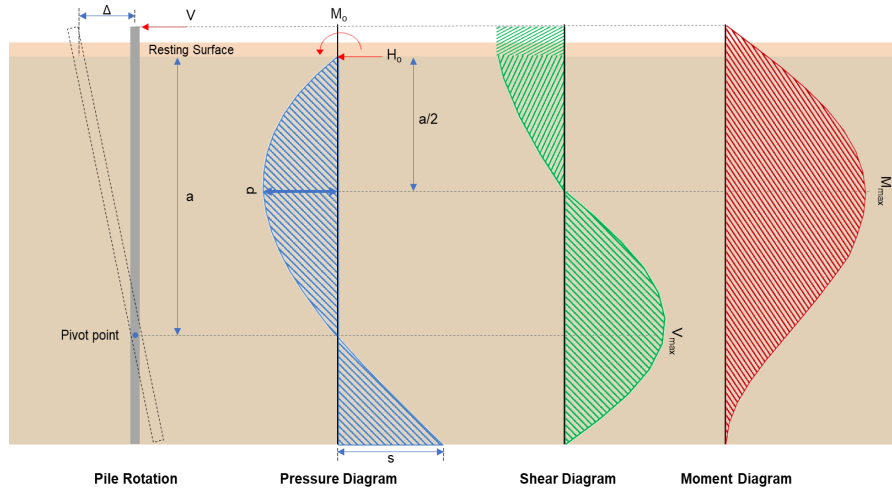
Status: **PASS**
Ratio: **0.000**

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

$$Ratio = \frac{(0.0011541 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.0010991$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.654 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.218 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.359 \text{ kipft}) + ((-0.654 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1197 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1197 \text{ kipft/ft})}{(-0.218 \text{ kip/ft})}$$

$$E = 23.485 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1197 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.218 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.1197 \text{ kipft/ft})) + (4 \times (-0.218 \text{ kip/ft}) \times (7 \text{ ft}))}$$

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

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

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

$$V_{max} = ((-0.218 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.3185 \text{ kip}$$

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

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

$$M_{max} = ((-0.218 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.485 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.485 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7634 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.696 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.009 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.003 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.045 \text{ kipft}) + ((-0.009 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.015 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015 \text{ kipft/ft})}{(-0.003 \text{ kip/ft})}$$

$$E = 5 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.003 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.015 \text{ kipft/ft})) + (4 \times (-0.003 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9483 \text{ ft}$$

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

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

$$V_{max} = ((-0.003 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9483 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9483 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.017341 \text{ kip}$$

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

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

$$M_{max} = ((-0.003 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9483 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.055047 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.929 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.962 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.962 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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. 10Ø: 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

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

$$\text{Ratio} = \frac{(6.929 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0046426$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.929 \text{ kip} \rightarrow 6929 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6929 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.719 \text{ kip}$$

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)

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

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

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

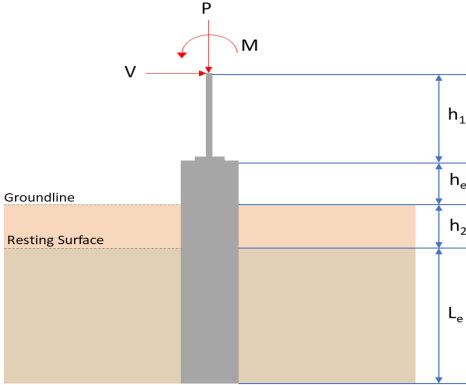
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(203.86 \text{ kip}), (82.719 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.719 \text{ kip}$</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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.719 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.578 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.3185 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.3185 \text{ kip})}{(78.578 \text{ kip})}$ $Ratio = 0.054959$ <p>Considering z-direction:</p> <p>$V_{max} = 0.017341 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.017341 \text{ kip})}{(78.578 \text{ kip})}$ $Ratio = 0.00022069$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <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: $M_{max} = 14.696 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.696 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.21629$	<p>Status: PASS Ratio: 0.220</p>
	<p>Considering z-direction: $M_{max} = 0.055047 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.055047 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.00081015$	<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>4.646</td> <td>6.562</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.379</td> <td>-0.632</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>0.094</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.650</td> <td>14.775</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)	4.646	6.562	V_x (kip)	-0.379	-0.632	V_z (kip)	0.013	0.018	M_x (kipft)	0.066	0.094	M_z (kipft)	8.650	14.775	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.379 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.12633 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(8.65 \text{ kipft}) + ((-0.379 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.8833 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.5736 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(6.5736 \text{ ft}), (1.5003 \text{ ft})]$$

$$L_{e,req} = 6.574 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.574 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.93914$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(4.646 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.65727 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.65727 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32864$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.12633 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.8833 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.8833 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.8833 \text{ kipft/ft})) + (4 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7657 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.8833 \text{ kipft/ft})) + (3 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.8833 \text{ kipft/ft})) + (2 \times (-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.27551 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.8833 \text{ kipft/ft})) + ((-0.12633 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.9391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7657 \text{ ft})}{2}$$

$$p_a = 0.35743 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27551 \text{ kip/ft}^2)}{(0.35743 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77081$$

Status: **PASS**
Ratio: **0.770**

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

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

$$\text{Ratio} = \frac{(0.9391 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89438$$

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

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,

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

$$a = \frac{(4 \times (0.022 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.0043333 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.022 \text{ kipft/ft})) + (4 \times (0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9461 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.022 \text{ kipft/ft})) + (3 \times (0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.022 \text{ kipft/ft})) + (2 \times (0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.0060813 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.022 \text{ kipft/ft})) + ((0.0043333 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.014298 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9461 \text{ ft})}{2}$$

$$p_a = 0.37095 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0060813 \text{ kip/ft}^2)}{(0.37095 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.016394$$

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

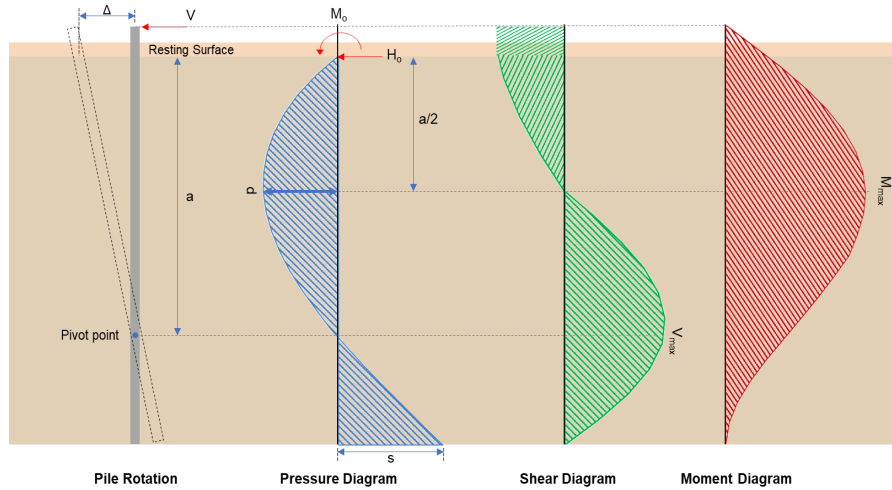
Status: **PASS**
Ratio: **0.020**

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

$$Ratio = \frac{(0.014298 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.013617$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.632 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.21067 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.775 \text{ kipft}) + ((-0.632 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.925 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.925 \text{ kipft/ft})}{(-0.21067 \text{ kip/ft})}$$

$$E = 23.378 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.925 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.21067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.925 \text{ kipft/ft})) + (4 \times (-0.21067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7637 \text{ ft}$$

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

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

$$V_{max} = ((-0.21067 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7637 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7637 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.1562 \text{ kip}$$

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

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

$$M_{max} = ((-0.21067 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(23.378 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.378 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7637 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.142 \text{ kipft}$$

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,

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

$$M_o = \frac{(0.094 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.031333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.031333 \text{ kipft/ft})}{(0.006 \text{ kip/ft})}$$

$$E = 5.2222 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.031333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.006 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.031333 \text{ kipft/ft})) + (4 \times (0.006 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.9419 \text{ ft}$$

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

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

$$V_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9419 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9419 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.035688 \text{ kip}$$

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

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

$$M_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(5.2222 \text{ ft})}{(7 \text{ ft})} + \frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2222 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.9419 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1136 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(6.562 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.973 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-44.973 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.562 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0043967$$

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 $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.562 \text{ kip} \rightarrow 6562 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6562 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.657 \text{ kip}$$

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

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.657 \text{ kip}), (237.06 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 82.657 \text{ kip}$</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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.657 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.538 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.1562 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.1562 \text{ kip})}{(78.538 \text{ kip})}$ $Ratio = 0.05292$ <p>Considering z-direction:</p> <p>$V_{max} = 0.035688 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.035688 \text{ kip})}{(78.538 \text{ kip})}$ $Ratio = 0.0004544$	<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: $M_{max} = 14.142 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.142 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.20813$	<p>Status: PASS Ratio: 0.210</p>
	<p>Considering z-direction: $M_{max} = 0.1136 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.1136 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.0016719$	<p>Status: PASS Ratio: 0.000</p>