

Your Project Calculations



Project Name: Elk Creek SPS 1 4x9S

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Elk%20Creek%20SPS%201%204x9S&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=1qfd7onE6lh2N8VZjy25qEsiey4WRjRdcqA4upmVyGhClk3mHZl68kF4PvH8Y2im

Array Specification

Product:	Beam
Unique ID:	4P-17-8TOP-XD-45-L-4Hx9W-EDDF
Duty Classification:	XD
Module Width:	44.60 in
Module Length:	89.20in
Number of Rows:	4
Number of Columns:	9
Total Number of Modules:	36
Desired Tilt Angle:	46
Front Edge Clearance:	15
Total Array Height at Tilt:	25.75 ft
Total Frame Length:	66.00 ft
Frame Weight:	5289 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	67.65 ft
Rail Length:	180.40 in
Rail Spacing:	3.76 ft
Rail Check:	

Support Specifications

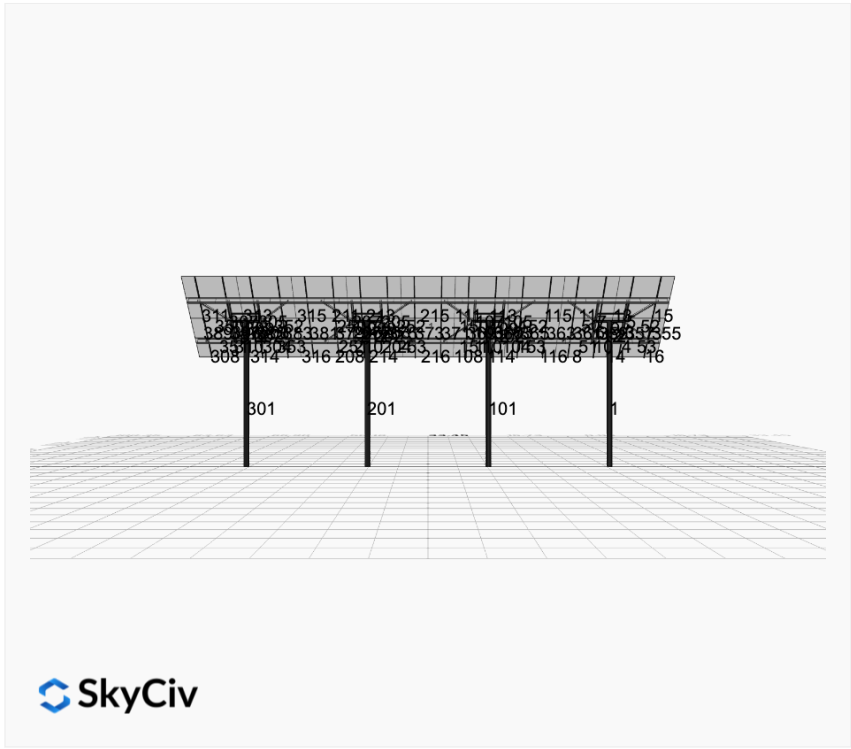
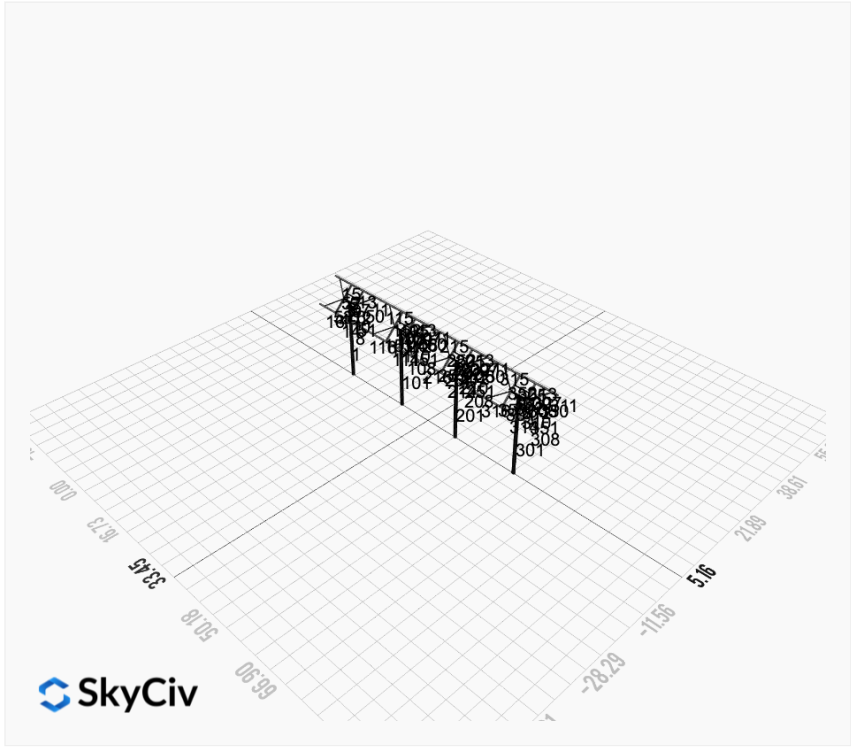
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	20.41 ft
Number of Poles:	4
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.25 ft Pile 2: 7.25 ft Pile 3: 7.25 ft Pile 4: 7.25 ft
Foundation Volume:	17.185 y ³
Foundation Result:	PASSED
Mount Twist:	0.214181 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	2222+22, College City, CA 95912, USA
Wind Speed:	100 mph
Snow Load:	176 psf
Design Uplift Pressure:	0.015331 ksf
Design Downforce Pressure:	-0.015331 ksf
Design Snow Pressure:	0.046449 ksf

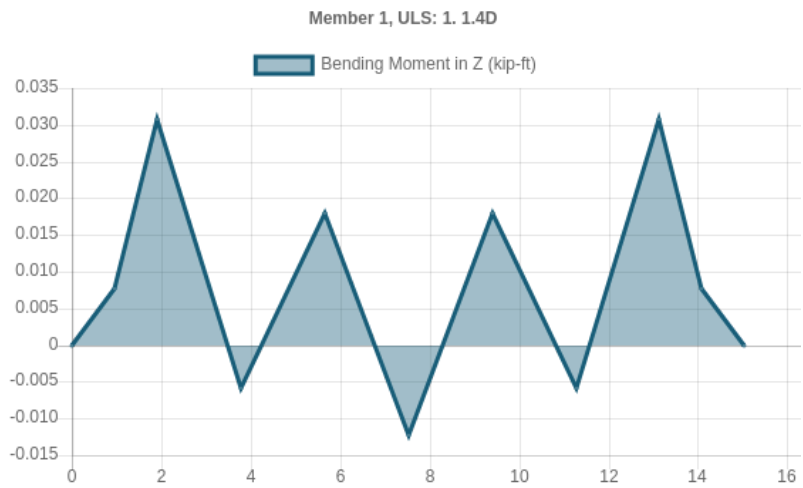


Rail Design Check

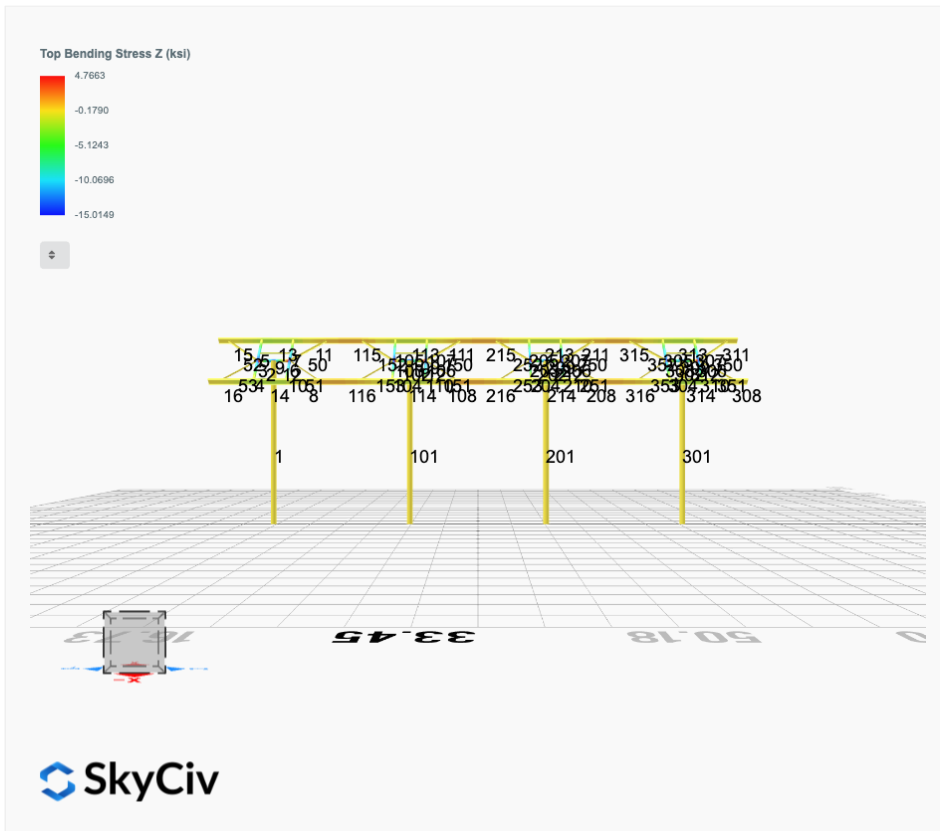
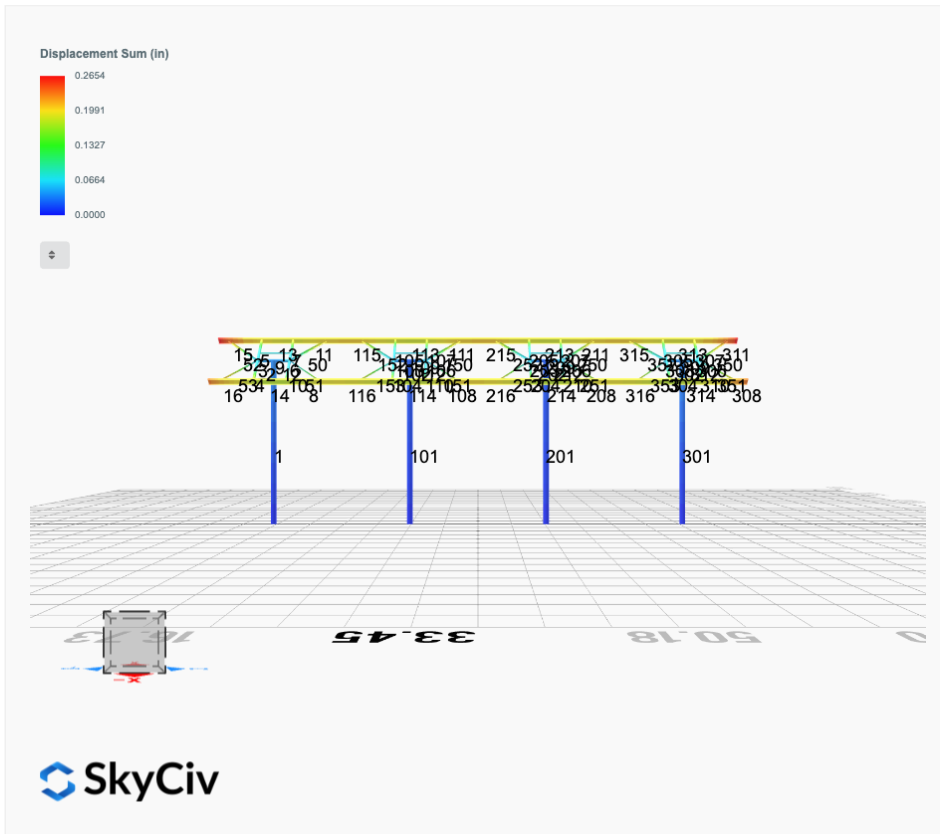
Rail Length: 15.033333333333333 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.758333333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.1213 kip/ft
Snow (Y): -0.1256 kip/ft
Wind uplift Case A: 0.0576 kip/ft
Wind downforce Case A: 0.0576 kip/ft
Dead (Panel load) (X): 0.0118 kip/ft
Dead (Panel load) (Y): -0.0122 kip/ft

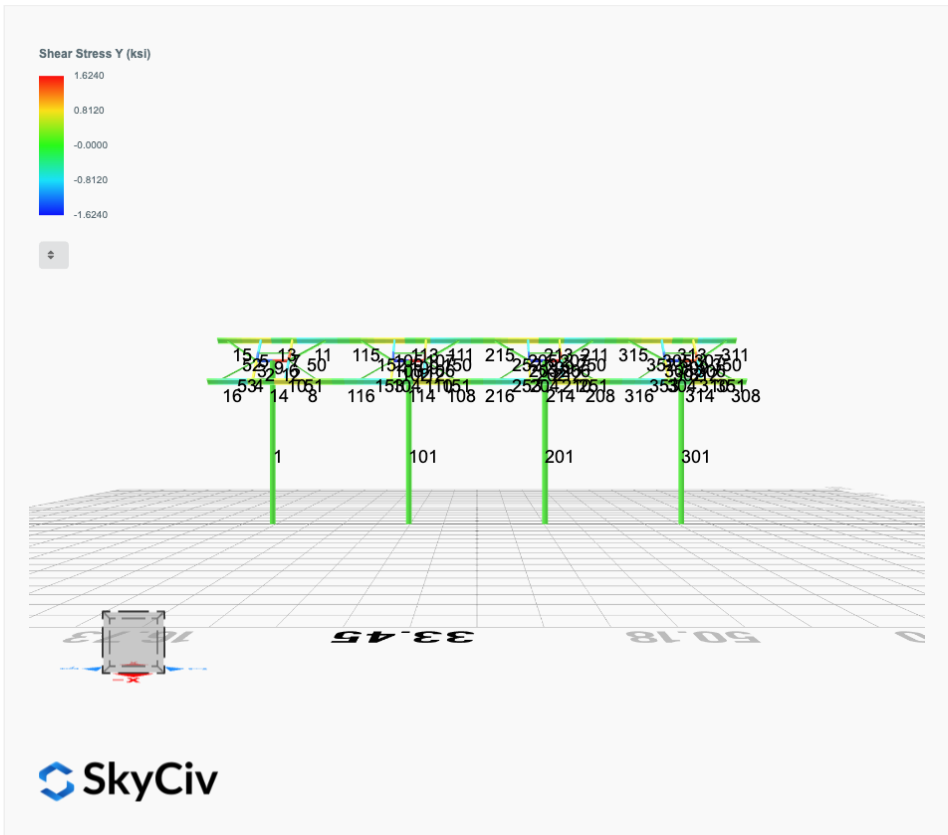
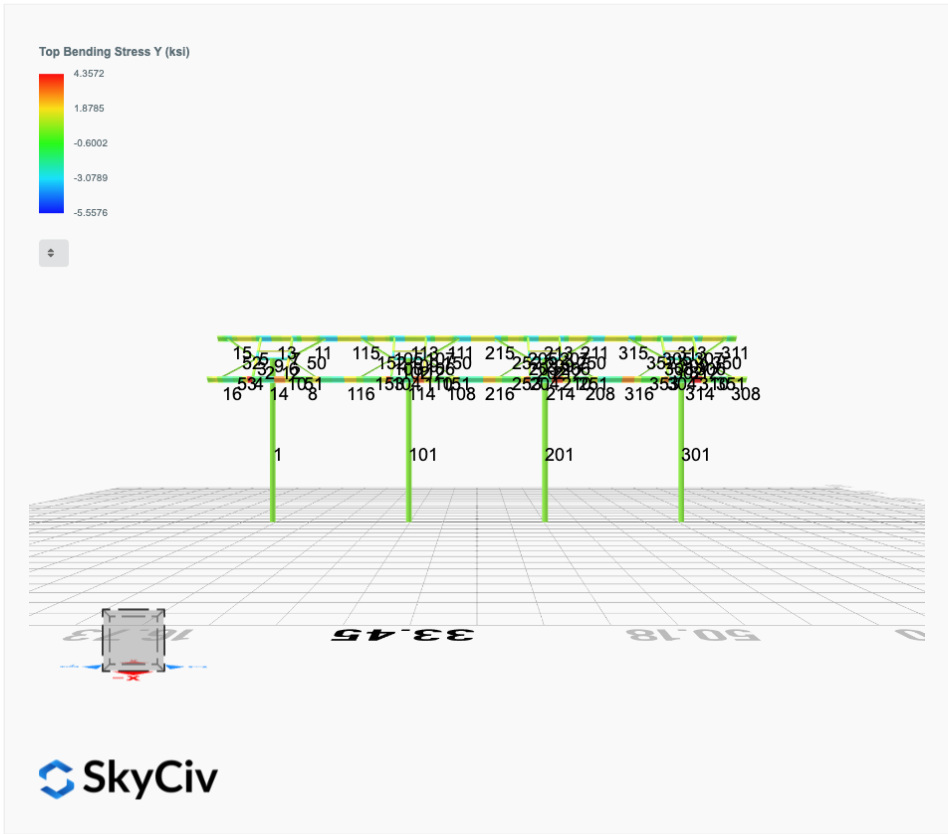


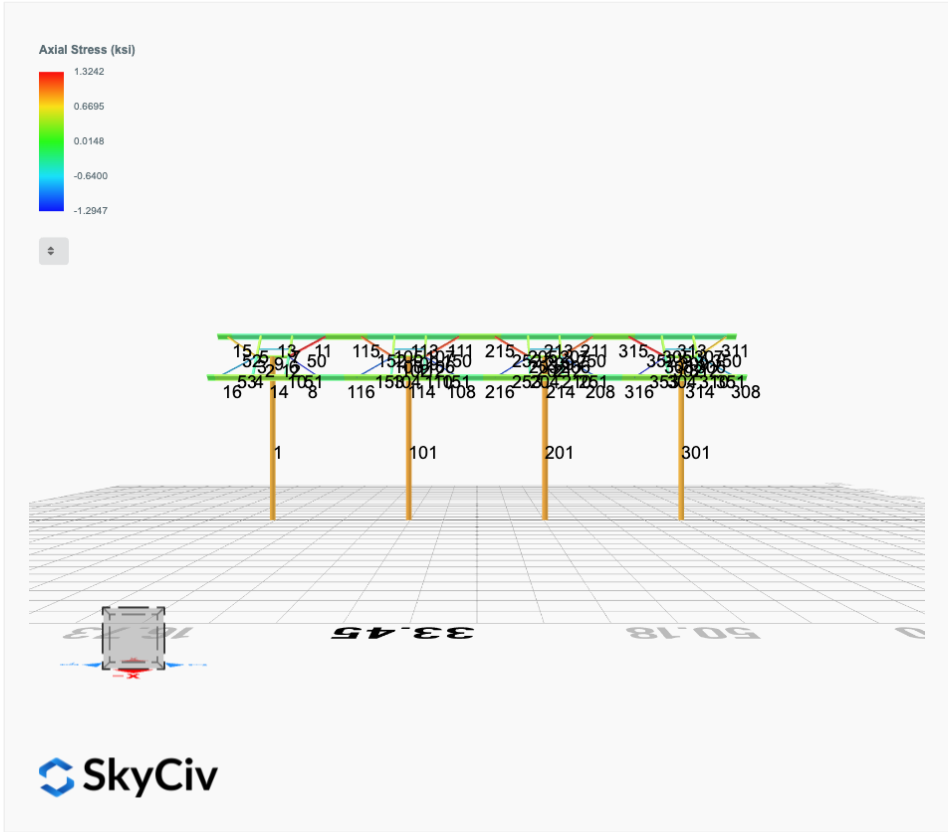
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	9.9016539	0.287	PASS
Material Yield	34.5	9.9016539	0.287	PASS
Material Strength	37	9.9016539	0.268	PASS



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	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 2. D + L	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 3. D + (S or Lr or R)	-0.0060	10.2716	-0.0040	-0.0244	0.1381	0.2107
ULS: 3. D + (S or Lr or R)	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0048	8.3143	-0.0033	-0.0200	0.1097	0.1690
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 5b. D + 0.7E	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0048	8.3143	-0.0033	-0.0200	0.1097	0.1690
ULS: 8. 0.6D + 0.7E	-0.0007	1.4654	-0.0006	-0.0040	0.0148	0.0264
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6809	4.0164	-0.0021	-0.0140	0.0189	35.2189
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6755	0.8692	0.0002	0.0019	0.0289	-33.3532
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2646	9.4948	-0.0041	-0.0255	0.1054	26.5502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0048	8.3143	-0.0033	-0.0200	0.1097	0.1690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2527	7.1344	-0.0023	-0.0136	0.1130	-24.8789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0048	8.3143	-0.0033	-0.0200	0.1097	0.1690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2610	3.6229	-0.0019	-0.0121	0.0203	26.4251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2563	1.2625	-0.0001	-0.0003	0.0278	-25.0039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0011	2.4424	-0.0011	-0.0066	0.0246	0.0440
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6805	3.0394	-0.0017	-0.0113	0.0090	35.2012
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0007	1.4654	-0.0006	-0.0040	0.0148	0.0264
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6759	-0.1078	0.0006	0.0045	0.0191	-33.3708
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0007	1.4654	-0.0006	-0.0040	0.0148	0.0264

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	16.7672
Shear X	-2.8083
Shear Z	-0.0077
Moment X	-0.0479
Moment Y (Twist)	0.2143
Moment Z	63.2250

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	10.2716
Shear X	-1.6809
Shear Z	-0.0041
Moment X	-0.0255
Moment Y (Twist)	0.1381
Moment Z	35.2189

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	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 2. D + L	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 3. D + (S or Lr or R)	0.0060	10.6839	0.0037	0.0239	-0.0086	-0.0009
ULS: 3. D + (S or Lr or R)	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0048	8.6394	0.0029	0.0190	-0.0068	0.0006
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 5b. D + 0.7E	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0048	8.6394	0.0029	0.0190	-0.0068	0.0006
ULS: 8. 0.6D + 0.7E	0.0007	1.5036	0.0004	0.0025	-0.0008	0.0031
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6838	4.1812	0.0035	0.0228	-0.0254	35.3198
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6893	0.8298	-0.0021	-0.0138	0.0218	-33.5476
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2590	9.8959	0.0050	0.0329	-0.0248	26.4865
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0048	8.6394	0.0029	0.0190	-0.0068	0.0006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2709	7.3823	0.0009	0.0055	0.0106	-25.1640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0048	8.6394	0.0029	0.0190	-0.0068	0.0006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2626	3.7624	0.0027	0.0181	-0.0194	26.4911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2673	1.2488	-0.0014	-0.0093	0.0160	-25.1594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0011	2.5059	0.0006	0.0041	-0.0014	0.0051
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6843	3.1789	0.0032	0.0211	-0.0249	35.3177
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0007	1.5036	0.0004	0.0025	-0.0008	0.0031
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6889	-0.1726	-0.0023	-0.0155	0.0223	-33.5497
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0007	1.5036	0.0004	0.0025	-0.0008	0.0031

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	17.4899
Shear X	-2.8138
Shear Z	0.0083
Moment X	0.0548
Moment Y (Twist)	0.0479
Moment Z	63.3069

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	10.6839
Shear X	-1.6893
Shear Z	0.0050
Moment X	0.0329
Moment Y (Twist)	0.0254
Moment Z	35.3198

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	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 2. D + L	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 3. D + (S or Lr or R)	0.0060	10.6839	-0.0037	-0.0239	0.0086	-0.0009
ULS: 3. D + (S or Lr or R)	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0048	8.6394	-0.0029	-0.0189	0.0068	0.0006
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 5b. D + 0.7E	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0048	8.6394	-0.0029	-0.0189	0.0068	0.0006
ULS: 8. 0.6D + 0.7E	0.0007	1.5036	-0.0004	-0.0025	0.0008	0.0031
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6838	4.1812	-0.0035	-0.0228	0.0254	35.3198
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6893	0.8298	0.0021	0.0138	-0.0217	-33.5476
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2590	9.8959	-0.0050	-0.0329	0.0248	26.4865
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0048	8.6394	-0.0029	-0.0189	0.0068	0.0006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2709	7.3823	-0.0009	-0.0055	-0.0106	-25.1640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0048	8.6394	-0.0029	-0.0189	0.0068	0.0006

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2626	3.7624	-0.0027	-0.0181	0.0194	26.4911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2673	1.2488	0.0014	0.0093	-0.0160	-25.1594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0011	2.5059	-0.0006	-0.0041	0.0014	0.0051
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6843	3.1789	-0.0032	-0.0211	0.0249	35.3177
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0007	1.5036	-0.0004	-0.0025	0.0008	0.0031
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6889	-0.1726	0.0023	0.0155	-0.0223	-33.5497
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0007	1.5036	-0.0004	-0.0025	0.0008	0.0031

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	17.4898
Shear X	-2.8138
Shear Z	-0.0083
Moment X	-0.0549
Moment Y (Twist)	0.0477
Moment Z	63.3071

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	10.6839
Shear X	-1.6893
Shear Z	-0.0050
Moment X	-0.0329
Moment Y (Twist)	0.0254
Moment Z	35.3198

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	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 2. D + L	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 3. D + (S or Lr or R)	-0.0060	10.2716	0.0040	0.0244	-0.1381	0.2107
ULS: 3. D + (S or Lr or R)	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0048	8.3143	0.0033	0.0200	-0.1097	0.1690
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 5b. D + 0.7E	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0048	8.3143	0.0033	0.0200	-0.1097	0.1690
ULS: 8. 0.6D + 0.7E	-0.0007	1.4654	0.0006	0.0040	-0.0148	0.0264
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6809	4.0164	0.0021	0.0140	-0.0189	35.2189
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6755	0.8692	-0.0002	-0.0019	-0.0289	-33.3532
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2646	9.4948	0.0041	0.0255	-0.1054	26.5502
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0048	8.3143	0.0033	0.0200	-0.1097	0.1690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2527	7.1344	0.0023	0.0136	-0.1130	-24.8789
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0048	8.3143	0.0033	0.0200	-0.1097	0.1690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2610	3.6229	0.0019	0.0122	-0.0203	26.4252
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2563	1.2625	0.0001	0.0003	-0.0278	-25.0039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0011	2.4424	0.0011	0.0066	-0.0246	0.0440
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6805	3.0394	0.0017	0.0113	-0.0090	35.2013
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0007	1.4654	0.0006	0.0040	-0.0148	0.0264
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6759	-0.1078	-0.0006	-0.0045	-0.0191	-33.3708
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0007	1.4654	0.0006	0.0040	-0.0148	0.0264

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	16.7674
Shear X	-2.8080
Shear Z	0.0077
Moment X	0.0479
Moment Y (Twist)	0.2142
Moment Z	63.2235

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

Result	Value (kip, kip-ft)
Axial	10.2716
Shear X	-1.6809
Shear Z	0.0041
Moment X	0.0255
Moment Y (Twist)	0.1381
Moment Z	35.2189

Project Details

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

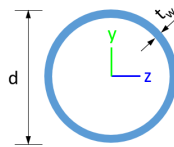


Design Input Information

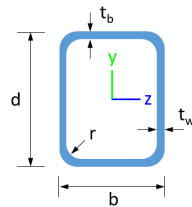
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

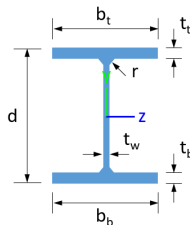
Section Dimensions



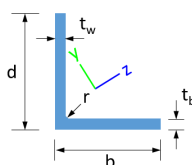
ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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



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



110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.17	6.46	56.26	44.91
114	159.30	116.35	32.39	6.46	56.26	44.91
115	159.30	61.62	33.81	6.46	56.26	44.91
116	159.30	61.62	34.12	6.46	56.26	44.91
150	41.27	8.45	1.63	0.88	15.23	10.15
151	41.27	8.45	1.63	0.88	15.23	10.15
152	41.27	8.45	1.63	0.88	15.23	10.15
153	41.27	8.45	1.63	0.88	15.23	10.15
201	377.97	61.92	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	142.47	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.17	6.46	56.26	44.91
214	159.30	116.35	32.09	6.46	56.26	44.91
215	159.30	61.62	33.20	6.46	56.26	44.91
216	159.30	61.62	33.81	6.46	56.26	44.91
250	41.27	8.45	1.63	0.88	15.23	10.15
251	41.27	8.45	1.63	0.88	15.23	10.15
252	41.27	8.45	1.63	0.88	15.23	10.15
253	41.27	8.45	1.63	0.88	15.23	10.15
301	377.97	61.92	83.29	83.29	113.39	113.39
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	119.60	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	119.60	46.90	6.46	56.26	44.91
312	251.01	246.00	27.16	27.16	75.30	75.30
313	159.30	116.35	33.92	6.46	56.26	44.91
314	159.30	116.35	33.62	6.46	56.26	44.91
315	159.30	61.62	35.66	6.46	56.26	44.91
316	159.30	61.62	35.66	6.46	56.26	44.91
350	41.27	8.45	1.63	0.88	15.23	10.15
351	41.27	8.45	1.63	0.88	15.23	10.15
352	41.27	8.45	1.63	0.88	15.23	10.15
353	41.27	8.45	1.63	0.88	15.23	10.15

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	0.271	0.759	0.001	0.025	0.000	0.836	#13	0.875	Not Required	Pass
2	0.007	0.454	0.118	0.105	0.019	0.570	#21	0.083	Not Required	Pass
3	0.010	0.577	0.178	0.057	0.073	0.760	#21	0.046	Not Required	Pass
4	0.010	0.570	0.068	0.056	0.011	0.643	#21	0.082	Not Required	Pass
5	0.010	0.358	0.034	0.056	0.008	0.396	#21	0.076	Not Required	Pass
6	0.008	0.571	0.198	0.056	0.078	0.771	#21	0.046	Not Required	Pass
7	0.008	0.354	0.021	0.056	0.003	0.379	#21	0.076	Not Required	Pass
8	0.007	0.045	0.052	0.033	0.012	0.090	#21	0.102	Not Required	Pass
9	0.031	0.048	0.055	0.002	0.001	0.103	#21	0.137	Not Required	Pass
10	0.009	0.563	0.094	0.056	0.016	0.661	#21	0.082	Not Required	Pass
11	0.006	0.045	0.051	0.033	0.013	0.089	#21	0.068	Not Required	Pass
12	0.007	0.450	0.115	0.108	0.019	0.558	#21	0.054	Not Required	Pass
13	0.007	0.185	0.095	0.045	0.017	0.281	#21	0.204	Not Required	Pass
14	0.012	0.188	0.114	0.044	0.018	0.286	#21	0.306	Not Required	Pass
15	0.007	0.057	0.049	0.025	0.013	0.100	#21	0.191	Not Required	Pass
16	0.009	0.057	0.039	0.025	0.013	0.087	#21	0.191	Not Required	Pass
50	0.271	0.009	0.004	0.004	0.002	0.281	#23	0.783	Not Required	Pass
51	0.054	0.005	0.015	0.003	0.002	0.041	#23	0.522	Not Required	Pass
52	0.181	0.009	0.004	0.003	0.002	0.102	#23	0.783	Not Required	Pass
53	0.036	0.005	0.015	0.002	0.002	0.033	#21	0.522	Not Required	Pass
101	0.282	0.760	0.001	0.025	0.000	0.841	#13	0.875	Not Required	Pass
102	0.008	0.467	0.115	0.111	0.018	0.580	#21	0.054	Not Required	Pass
103	0.009	0.596	0.196	0.059	0.078	0.796	#21	0.046	Not Required	Pass
104	0.010	0.587	0.085	0.058	0.014	0.677	#21	0.082	Not Required	Pass
105	0.009	0.370	0.030	0.058	0.006	0.404	#21	0.076	Not Required	Pass
106	0.009	0.599	0.197	0.059	0.079	0.801	#21	0.046	Not Required	Pass
107	0.009	0.372	0.028	0.059	0.005	0.405	#21	0.076	Not Required	Pass
108	0.006	0.059	0.033	0.032	0.011	0.095	#21	0.102	Not Required	Pass
109	0.032	0.036	0.048	0.001	0.000	0.100	#21	0.137	Not Required	Pass
110	0.009	0.591	0.085	0.059	0.014	0.680	#21	0.082	Not Required	Pass
111	0.006	0.059	0.036	0.032	0.012	0.096	#21	0.102	Not Required	Pass
112	0.008	0.472	0.117	0.112	0.019	0.585	#21	0.054	Not Required	Pass
113	0.006	0.142	0.082	0.043	0.016	0.218	#21	0.306	Not Required	Pass
114	0.011	0.142	0.109	0.043	0.017	0.227	#21	0.306	Not Required	Pass
115	0.012	0.103	0.097	0.031	0.017	0.164	#21	0.570	Not Required	Pass
116	0.016	0.103	0.095	0.030	0.017	0.164	#21	0.570	Not Required	Pass
150	0.253	0.009	0.004	0.003	0.002	0.263	#23	0.783	Not Required	Pass
151	0.050	0.005	0.015	0.003	0.002	0.039	#23	0.522	Not Required	Pass
152	0.246	0.009	0.004	0.003	0.002	0.256	#21	0.783	Not Required	Pass
153	0.048	0.005	0.015	0.003	0.002	0.038	#21	0.522	Not Required	Pass
201	0.282	0.760	0.001	0.025	0.000	0.841	#13	0.875	Not Required	Pass
202	0.008	0.472	0.117	0.112	0.019	0.585	#21	0.054	Not Required	Pass
203	0.009	0.599	0.197	0.059	0.079	0.801	#21	0.046	Not Required	Pass
204	0.009	0.591	0.085	0.059	0.014	0.680	#21	0.082	Not Required	Pass
205	0.009	0.372	0.028	0.059	0.005	0.405	#21	0.076	Not Required	Pass
206	0.009	0.596	0.196	0.059	0.078	0.796	#21	0.046	Not Required	Pass
207	0.009	0.370	0.030	0.058	0.006	0.404	#21	0.076	Not Required	Pass
208	0.006	0.052	0.033	0.030	0.011	0.080	#21	0.102	Not Required	Pass
209	0.032	0.036	0.048	0.001	0.000	0.100	#21	0.137	Not Required	Pass
210	0.010	0.587	0.085	0.058	0.014	0.677	#21	0.082	Not Required	Pass
211	0.006	0.052	0.042	0.031	0.012	0.082	#21	0.102	Not Required	Pass
212	0.008	0.467	0.115	0.111	0.018	0.580	#21	0.054	Not Required	Pass

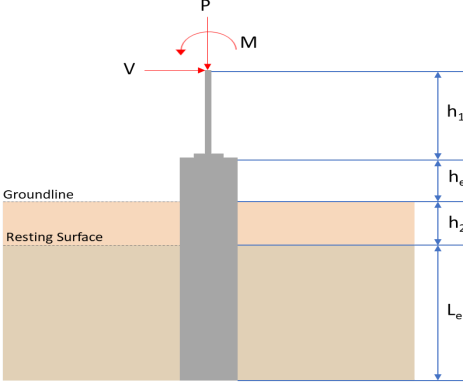
213	0.006	0.142	0.082	0.043	0.016	0.218	#21	0.306	Not Required	Pass
214	0.011	0.142	0.109	0.043	0.017	0.227	#21	0.306	Not Required	Pass
215	0.014	0.117	0.077	0.032	0.016	0.176	#21	0.570	Not Required	Pass
216	0.014	0.118	0.072	0.032	0.016	0.173	#21	0.570	Not Required	Pass
250	0.246	0.009	0.004	0.003	0.002	0.256	#21	0.783	Not Required	Pass
251	0.048	0.005	0.015	0.002	0.002	0.038	#21	0.522	Not Required	Pass
252	0.253	0.009	0.004	0.003	0.002	0.263	#23	0.783	Not Required	Pass
253	0.050	0.005	0.015	0.003	0.002	0.039	#23	0.522	Not Required	Pass
301	0.271	0.759	0.001	0.025	0.000	0.836	#13	0.875	Not Required	Pass
302	0.007	0.450	0.115	0.108	0.019	0.558	#21	0.054	Not Required	Pass
303	0.008	0.571	0.198	0.056	0.078	0.771	#21	0.046	Not Required	Pass
304	0.009	0.563	0.094	0.056	0.016	0.661	#21	0.082	Not Required	Pass
305	0.008	0.354	0.021	0.056	0.003	0.379	#21	0.076	Not Required	Pass
306	0.010	0.577	0.178	0.057	0.073	0.760	#21	0.046	Not Required	Pass
307	0.010	0.358	0.034	0.056	0.008	0.396	#21	0.076	Not Required	Pass
308	0.009	0.057	0.039	0.025	0.013	0.087	#21	0.287	Not Required	Pass
309	0.031	0.048	0.055	0.002	0.001	0.103	#21	0.137	Not Required	Pass
310	0.010	0.569	0.068	0.056	0.011	0.643	#21	0.082	Not Required	Pass
311	0.007	0.057	0.049	0.025	0.013	0.100	#21	0.191	Not Required	Pass
312	0.007	0.454	0.118	0.105	0.019	0.570	#21	0.083	Not Required	Pass
313	0.007	0.185	0.095	0.045	0.017	0.281	#21	0.204	Not Required	Pass
314	0.012	0.188	0.114	0.044	0.018	0.286	#21	0.306	Not Required	Pass
315	0.012	0.101	0.097	0.033	0.017	0.163	#21	0.380	Not Required	Pass
316	0.016	0.099	0.095	0.033	0.017	0.162	#21	0.570	Not Required	Pass
350	0.181	0.009	0.004	0.003	0.002	0.102	#21	0.783	Not Required	Pass
351	0.036	0.005	0.015	0.002	0.002	0.033	#21	0.522	Not Required	Pass
352	0.271	0.009	0.004	0.003	0.002	0.281	#23	0.783	Not Required	Pass
353	0.054	0.005	0.015	0.003	0.002	0.041	#23	0.522	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

OK
NG

Capacity is provided
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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 7.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to 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 1102 1193 1193"> <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 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.272</td> <td>16.767</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.681</td> <td>-2.808</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.004</td> <td>-0.008</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.025</td> <td>-0.048</td> </tr> <tr> <td>M_z (kipft)</td> <td>35.219</td> <td>63.225</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	10.272	16.767	V_x (kip)	-1.681	-2.808	V_z (kip)	-0.004	-0.008	M_x (kipft)	-0.025	-0.048	M_z (kipft)	35.219	63.225	
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)	10.272	16.767																										
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V_z (kip)	-0.004	-0.008																										
M_x (kipft)	-0.025	-0.048																										
M_z (kipft)	35.219	63.225																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.681 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26768 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(35.219 \text{ kipft}) + ((-1.681 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6081 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.9582 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.004 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00063694 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.025 \text{ kipft}) + ((-0.004 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0039809 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.66411 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9582 \text{ ft}), (0.66411 \text{ ft})]$$

$$L_{e,req} = 6.958 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.958 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.95972$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(10.272 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.642 \text{ kip/ft}^2$$

$$q = 0.042 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.642 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.321$$

Status: **PASS**
Ratio: **0.320**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6081 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6081 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.6081 \text{ kipft/ft})) + (4 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9466 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6081 \text{ kipft/ft})) + (3 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.6081 \text{ kipft/ft})) + (2 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.30417 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6081 \text{ kipft/ft})) + ((-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0588 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9466 \text{ ft})}{2}$$

$$p_a = 0.37099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30417 \text{ kip/ft}^2)}{(0.37099 \text{ kip/ft}^2)}$$

$$Ratio = 0.81988$$

p_a - Allowable lateral soil pressure at depth L_e ,

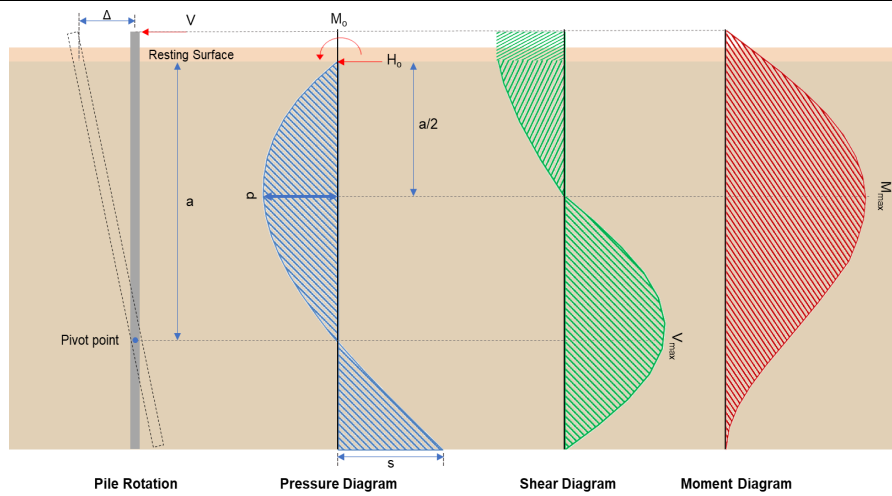
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0588 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97362$	Status: PASS Ratio: 0.970
	<p>Considering z-direction:</p> <p>$H_o = -0.00063694 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0039809 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0039809 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0039809 \text{ kipft/ft})) + (4 \times (-0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))}$ $a = 5.0968 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0039809 \text{ kipft/ft})) + (3 \times (-0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0039809 \text{ kipft/ft})) + (2 \times (-0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$ $p = 0.000022587 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0039809 \text{ kipft/ft})) + ((-0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$ $s = 0.00038171 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0968 \text{ ft})}{2}$ $p_a = 0.38226 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.000022587 \text{ kip/ft}^2)}{(0.38226 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.000059089$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(0.00038171 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.000351$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.808 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(63.225 \text{ kipft}) + ((-2.808 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.068 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.068 \text{ kipft/ft})}{(-0.44713 \text{ kip/ft})}$$

$$E = 22.516 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.068 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44713 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (10.068 \text{ kipft/ft})) + (4 \times (-0.44713 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9401 \text{ ft}$$

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

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

$$V_{max} = ((-0.44713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.516 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9401 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.516 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9401 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.019 \text{ kip}$$

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

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

$$M_{max} = ((-0.44713 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(22.516 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.516 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (22.516 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 38.757 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.048 \text{ kipft}) + ((-0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0076433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0076433 \text{ kipft/ft})}{(-0.0012739 \text{ kip/ft})}$$

$$E = 6 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0076433 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0076433 \text{ kipft/ft})) + (4 \times (-0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1029 \text{ ft}$$

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

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

$$V_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1029 \text{ ft})}{(7.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1029 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.010834 \text{ kip}$$

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

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

$$M_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(6 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.035954 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(16.767 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.039 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.039 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #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

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.767 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0062676$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.767 \text{ kip} \rightarrow 16767 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(16767 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.72 \text{ kip}$$

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

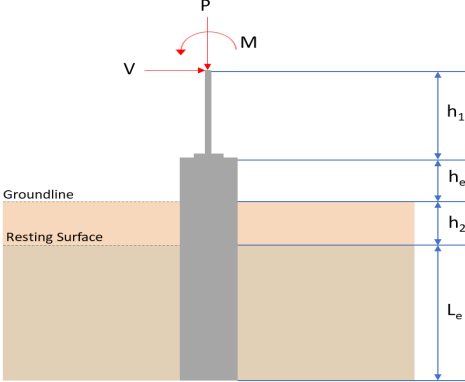
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.72 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.72 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.72 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.55 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.019 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.019 \text{ kip})}{(111.55 \text{ kip})}$ $\text{Ratio} = 0.098778$ <p>Considering z-direction:</p> <p>$V_{max} = 0.010834 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.010834 \text{ kip})}{(111.55 \text{ kip})}$ $\text{Ratio} = 0.000097121$	<p>Status: PASS Ratio: 0.100</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 38.757 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.757 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.15528$	<p>Status: PASS Ratio: 0.160</p>
	<p>Considering z-direction: $M_{max} = 0.035954 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.035954 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00014405$	<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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 7.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to 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 1102 1193 1191"> <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 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.684</td> <td>17.490</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.689</td> <td>-2.814</td> </tr> <tr> <td>V_z (kip)</td> <td>0.005</td> <td>0.008</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.033</td> <td>0.055</td> </tr> <tr> <td>M_z (kipft)</td> <td>35.320</td> <td>63.307</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	10.684	17.490	V_x (kip)	-1.689	-2.814	V_z (kip)	0.005	0.008	M_x (kipft)	0.033	0.055	M_z (kipft)	35.320	63.307	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.689 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26895 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(35.32 \text{ kipft}) + ((-1.689 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6242 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.9629 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0052548 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.77025 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9629 \text{ ft}), (0.77025 \text{ ft})]$$

$$L_{e,req} = 6.963 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.963 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.96041$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(10.684 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.66775 \text{ kip/ft}^2$$

$$q = 0.001 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.66775 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.33387$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26895 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6242 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6242 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.6242 \text{ kipft/ft})) + (4 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9468 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6242 \text{ kipft/ft})) + (3 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.6242 \text{ kipft/ft})) + (2 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.30481 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6242 \text{ kipft/ft})) + ((-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0614 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9468 \text{ ft})}{2}$$

$$p_a = 0.37101 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30481 \text{ kip/ft}^2)}{(0.37101 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82158$$

p_a - Allowable lateral soil pressure at depth L_e ,

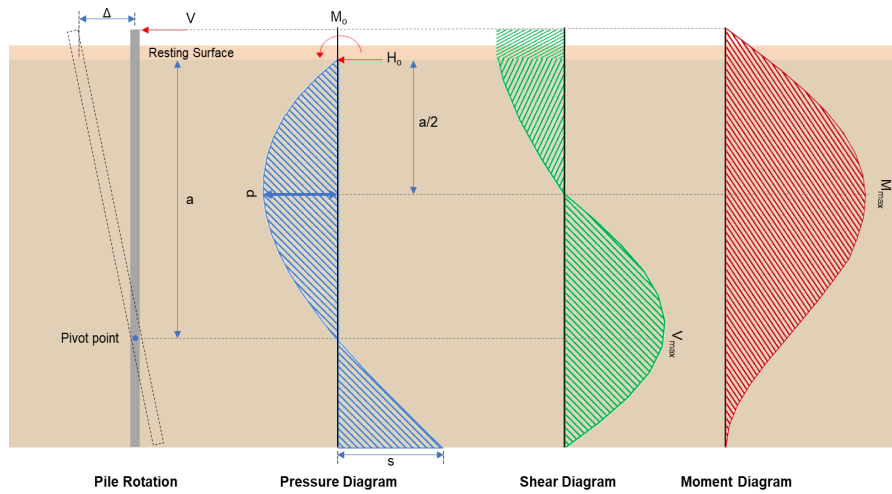
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0614 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97602$	Status: PASS Ratio: 0.980
	<p>Considering z-direction:</p> <p>$H_o = 0.00079618 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0052548 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0052548 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0052548 \text{ kipft/ft})) + (4 \times (0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))}$ $a = 5.0887 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0052548 \text{ kipft/ft})) + (3 \times (0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0052548 \text{ kipft/ft})) + (2 \times (0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$ $p = 0.00076788 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0052548 \text{ kipft/ft})) + ((0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$ $s = 0.0018586 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0887 \text{ ft})}{2}$ $p_a = 0.38166 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00076788 \text{ kip/ft}^2)}{(0.38166 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.002012$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(0.0018586 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.001709$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.814 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(63.307 \text{ kipft}) + ((-2.814 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.081 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.081 \text{ kipft/ft})}{(-0.44809 \text{ kip/ft})}$$

$$E = 22.497 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.081 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44809 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (10.081 \text{ kipft/ft})) + (4 \times (-0.44809 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9402 \text{ ft}$$

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

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

$$V_{max} = ((-0.44809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9402 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9402 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.034 \text{ kip}$$

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

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

$$M_{max} = ((-0.44809 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(22.497 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 38.81 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.008758 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.008758 \text{ kipft/ft})}{(0.0012739 \text{ kip/ft})}$$

$$E = 6.875 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0827 \text{ ft}$$

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

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

$$V_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0827 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0827 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.011917 \text{ kip}$$

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

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

$$M_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(6.875 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.039875 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.49 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.015 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.015 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #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

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.49 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0065379$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.49 \text{ kip} \rightarrow 17490 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(17490 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.82 \text{ kip}$$

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

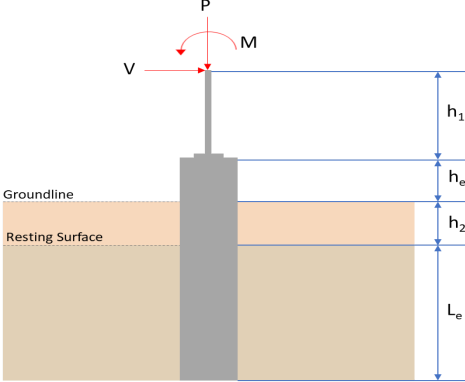
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.82 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.82 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.82 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.61 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.034 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.034 \text{ kip})}{(111.61 \text{ kip})}$ $\text{Ratio} = 0.098859$ <p>Considering z-direction:</p> <p>$V_{max} = 0.011917 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.011917 \text{ kip})}{(111.61 \text{ kip})}$ $\text{Ratio} = 0.00010677$	<p>Status: PASS Ratio: 0.100</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 38.81 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.81 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.15549$	<p>Status: PASS Ratio: 0.160</p>
	<p>Considering z-direction: $M_{max} = 0.039875 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.039875 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00015975$	<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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 7.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to 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 1102 1193 1191"> <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 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.684</td> <td>17.490</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.689</td> <td>-2.814</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.005</td> <td>-0.008</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.033</td> <td>-0.055</td> </tr> <tr> <td>M_z (kipft)</td> <td>35.320</td> <td>63.307</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	10.684	17.490	V_x (kip)	-1.689	-2.814	V_z (kip)	-0.005	-0.008	M_x (kipft)	-0.033	-0.055	M_z (kipft)	35.320	63.307	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.689 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26895 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(35.32 \text{ kipft}) + ((-1.689 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6242 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.9629 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((-0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0052548 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.72753 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9629 \text{ ft}), (0.72753 \text{ ft})]$$

$$L_{e,req} = 6.963 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.963 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.96041$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(10.684 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.66775 \text{ kip/ft}^2$$

$$q = 0.001 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.66775 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.33387$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26895 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6242 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6242 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.6242 \text{ kipft/ft})) + (4 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9468 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6242 \text{ kipft/ft})) + (3 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.6242 \text{ kipft/ft})) + (2 \times (-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.30481 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6242 \text{ kipft/ft})) + ((-0.26895 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0614 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9468 \text{ ft})}{2}$$

$$p_a = 0.37101 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.30481 \text{ kip/ft}^2)}{(0.37101 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82158$$

p_a - Allowable lateral soil pressure at depth L_e ,

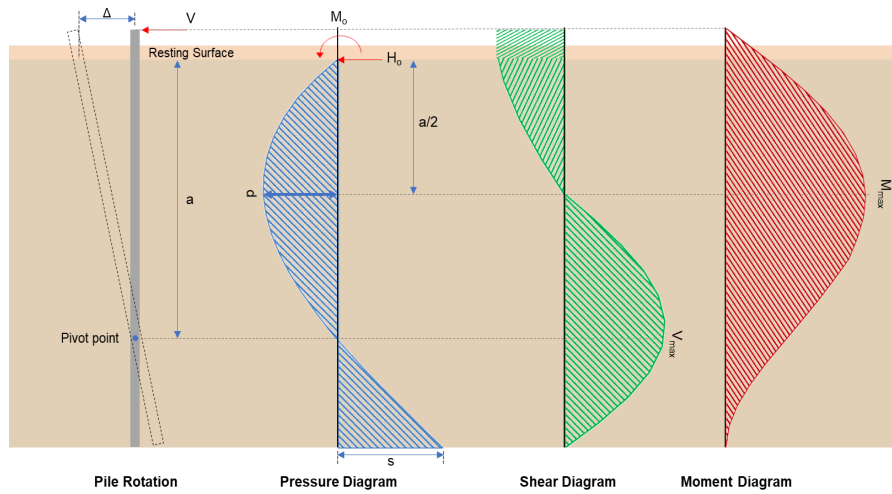
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0614 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97602$	<p>Status: PASS Ratio: 0.980</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.00079618 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0052548 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0052548 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0052548 \text{ kipft/ft})) + (4 \times (-0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))}$ $a = 5.0887 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0052548 \text{ kipft/ft})) + (3 \times (-0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0052548 \text{ kipft/ft})) + (2 \times (-0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$ $p = 0.000046347 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0052548 \text{ kipft/ft})) + ((-0.00079618 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$ $s = 0.00054076 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0887 \text{ ft})}{2}$ $p_a = 0.38166 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.000046347 \text{ kip/ft}^2)}{(0.38166 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00012144$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.000</p>

$$\text{Ratio} = \frac{(0.00054076 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00049725$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.814 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44809 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(63.307 \text{ kipft}) + ((-2.814 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.081 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.081 \text{ kipft/ft})}{(-0.44809 \text{ kip/ft})}$$

$$E = 22.497 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.081 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44809 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (10.081 \text{ kipft/ft})) + (4 \times (-0.44809 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9402 \text{ ft}$$

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

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

$$V_{max} = ((-0.44809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9402 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9402 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.034 \text{ kip}$$

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

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

$$M_{max} = ((-0.44809 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(22.497 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.497 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9402 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 38.81 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((-0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.008758 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.008758 \text{ kipft/ft})}{(-0.0012739 \text{ kip/ft})}$$

$$E = 6.875 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (-0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0827 \text{ ft}$$

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

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

$$V_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0827 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0827 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.011917 \text{ kip}$$

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

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

$$M_{max} = ((-0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(6.875 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (6.875 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0827 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.039875 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.49 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.015 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.015 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #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

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(17.49 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0065379$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.49 \text{ kip} \rightarrow 17490 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(17490 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.82 \text{ kip}$$

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

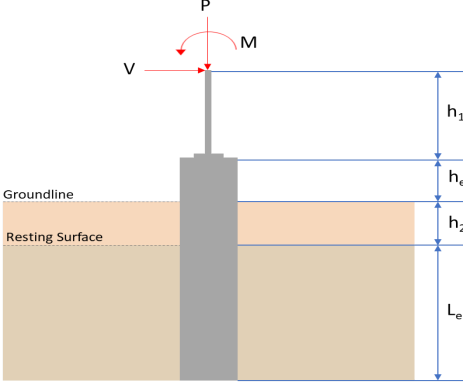
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.82 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.82 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.82 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.61 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.034 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.034 \text{ kip})}{(111.61 \text{ kip})}$ $\text{Ratio} = 0.098859$ <p>Considering z-direction:</p> <p>$V_{max} = 0.011917 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.011917 \text{ kip})}{(111.61 \text{ kip})}$ $\text{Ratio} = 0.00010677$	<p>Status: PASS Ratio: 0.100</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 38.81 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.81 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.15549$	<p>Status: PASS Ratio: 0.160</p>
	<p>Considering z-direction: $M_{max} = 0.039875 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.039875 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00015975$	<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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 7.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to 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 1102 1193 1191"> <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 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.272</td> <td>16.767</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.681</td> <td>-2.808</td> </tr> <tr> <td>V_z (kip)</td> <td>0.004</td> <td>0.008</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.025</td> <td>0.048</td> </tr> <tr> <td>M_z (kipft)</td> <td>35.219</td> <td>63.223</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	10.272	16.767	V_x (kip)	-1.681	-2.808	V_z (kip)	0.004	0.008	M_x (kipft)	0.025	0.048	M_z (kipft)	35.219	63.223	
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M_z (kipft)	35.219	63.223																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.681 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.26768 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(35.219 \text{ kipft}) + ((-1.681 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6081 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.9582 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.004 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00063694 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.025 \text{ kipft}) + ((0.004 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0039809 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.70159 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.9582 \text{ ft}), (0.70159 \text{ ft})]$$

$$L_{e,req} = 6.958 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.958 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.95972$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(10.272 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.642 \text{ kip/ft}^2$$

$$q = 0.042 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.642 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.321$$

Status: **PASS**
Ratio: **0.320**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26768 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6081 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6081 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.6081 \text{ kipft/ft})) + (4 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9466 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6081 \text{ kipft/ft})) + (3 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.6081 \text{ kipft/ft})) + (2 \times (-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.30417 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6081 \text{ kipft/ft})) + ((-0.26768 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0588 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.9466 \text{ ft})}{2}$$

$$p_a = 0.37099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30417 \text{ kip/ft}^2)}{(0.37099 \text{ kip/ft}^2)}$$

$$Ratio = 0.81988$$

p_a - Allowable lateral soil pressure at depth L_e ,

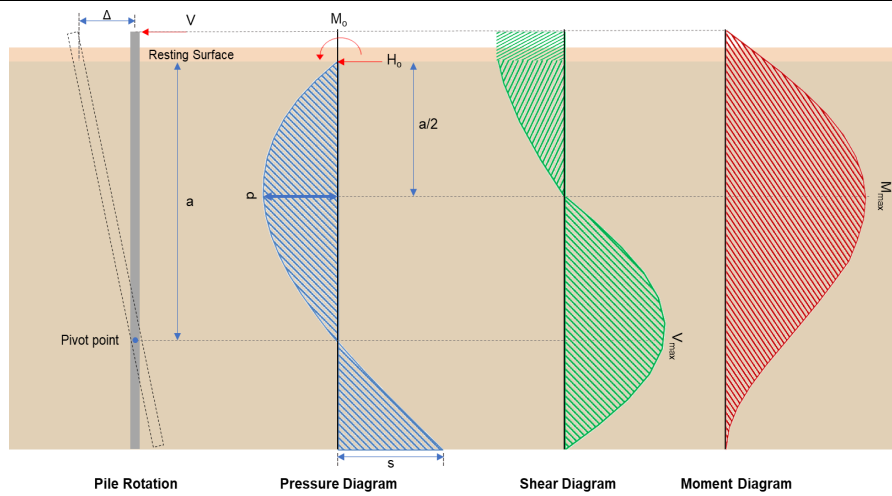
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.0588 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.97362$	Status: PASS Ratio: 0.970
	<p>Considering z-direction:</p> <p>$H_o = 0.00063694 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0039809 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0039809 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0039809 \text{ kipft/ft})) + (4 \times (0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))}$ $a = 5.0968 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0039809 \text{ kipft/ft})) + (3 \times (0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0039809 \text{ kipft/ft})) + (2 \times (0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$ $p = 0.00059739 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0039809 \text{ kipft/ft})) + ((0.00063694 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$ $s = 0.001436 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(5.0968 \text{ ft})}{2}$ $p_a = 0.38226 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00059739 \text{ kip/ft}^2)}{(0.38226 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0015628$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$ $p_s = 1.0875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$Ratio = \frac{(0.001436 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0013204$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.808 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.44713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(63.223 \text{ kipft}) + ((-2.808 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.067 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.067 \text{ kipft/ft})}{(-0.44713 \text{ kip/ft})}$$

$$E = 22.515 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.067 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.44713 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (10.067 \text{ kipft/ft})) + (4 \times (-0.44713 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.9401 \text{ ft}$$

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

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

$$V_{max} = ((-0.44713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.515 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9401 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.515 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9401 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.018 \text{ kip}$$

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

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

$$M_{max} = ((-0.44713 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(22.515 \text{ ft})}{(7.25 \text{ ft})} + \frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.515 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.515 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.9401 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 38.756 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0012739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.048 \text{ kipft}) + ((0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0076433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0076433 \text{ kipft/ft})}{(0.0012739 \text{ kip/ft})}$$

$$E = 6 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0076433 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0076433 \text{ kipft/ft})) + (4 \times (0.0012739 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1029 \text{ ft}$$

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

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

$$V_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1029 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1029 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.010834 \text{ kip}$$

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

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

$$M_{max} = ((0.0012739 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(6 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (6 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1029 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.035954 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(16.767 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.039 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.039 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

$$A_{min} = 4.1472 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$\text{Ratio} = 0.96556$$

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

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: Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #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

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.767 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0062676$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 296.21 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.767 \text{ kip} \rightarrow 16767 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(16767 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.72 \text{ kip}$$

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.72 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.72 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.72 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.55 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.018 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.018 \text{ kip})}{(111.55 \text{ kip})}$ $\text{Ratio} = 0.098775$ <p>Considering z-direction:</p> <p>$V_{max} = 0.010834 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.010834 \text{ kip})}{(111.55 \text{ kip})}$ $\text{Ratio} = 0.000097121$	<p>Status: PASS Ratio: 0.100</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 38.756 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.756 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.15527$	<p>Status: PASS Ratio: 0.160</p>
	<p>Considering z-direction: $M_{max} = 0.035954 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.035954 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00014405$	<p>Status: PASS Ratio: 0.000</p>