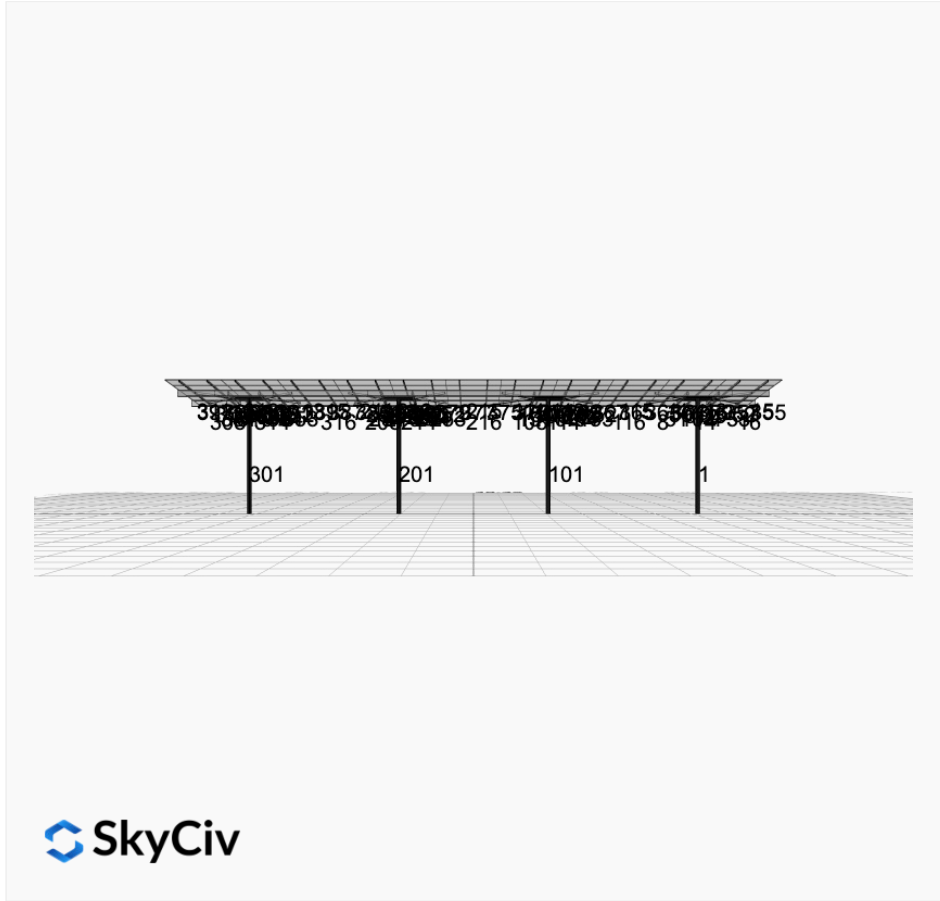


Project Name: Columbus|Lozza 5x11 - V1|b **Date:** Fri Nov 29 2024
Location: 420 Columbus Ave, Valhalla, NY 10595, USA **Number of Modules:** 55
Unique ID: 4P-19.75-6TOP-XD-57-L-5Hx11W-STRUTS-83A5 **Number of Poles:** 4
83A5 **Date Sold:**
Dealer: _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	76.63 ft
Winter Tilt Angle	7.5
Front Edge Clearance	14 ft

MT Solar Bill of Materials (4P-19.75-6TOP-XD-57-L-5Hx11W-STRUTS-83A5)

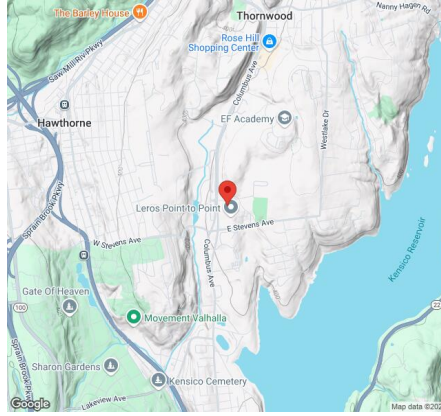
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-57	57IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	11

Rail Bill of Materials

Part	Qty
Rails (223in)	22
Rail Attachment	88

Part	Qty
Module Mid Clamp	88
Module End Clamp	44
Ground Lug	11

Site Details:



Site Address: 420 Columbus Ave, Valhalla, NY 10595, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	11
Total Number of Modules:	55
Winter Tilt Angle:	7.5
Front Edge Clearance:	14
Total Array Height at Tilt:	16.45 ft
Total Frame Length:	76.25 ft
Frame Weight:	5524 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	76.63 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	15.23 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.50 ft Pile 2: 4.50 ft Pile 3: 4.50 ft Pile 4: 4.50 ft
Foundation Volume:	10.667 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	420 Columbus Ave, Valhalla, NY 10595, USA
Wind Speed:	90 mph

Snow Load:

30 psf

Design Disclaimer

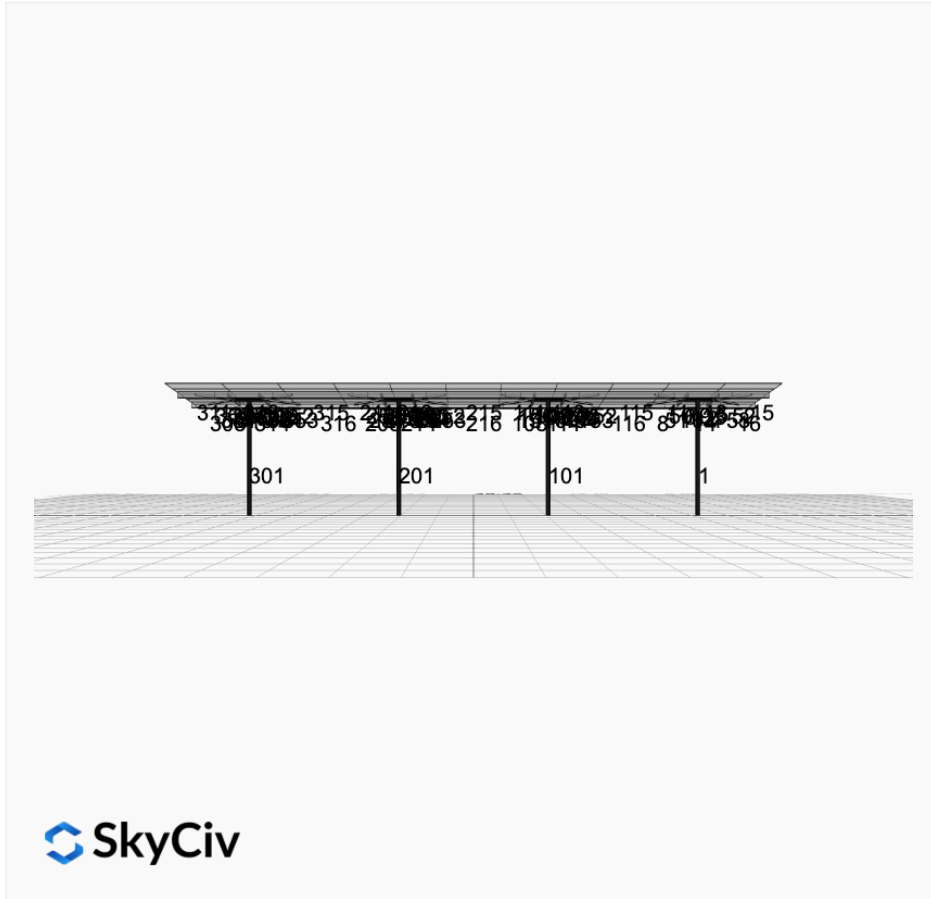
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

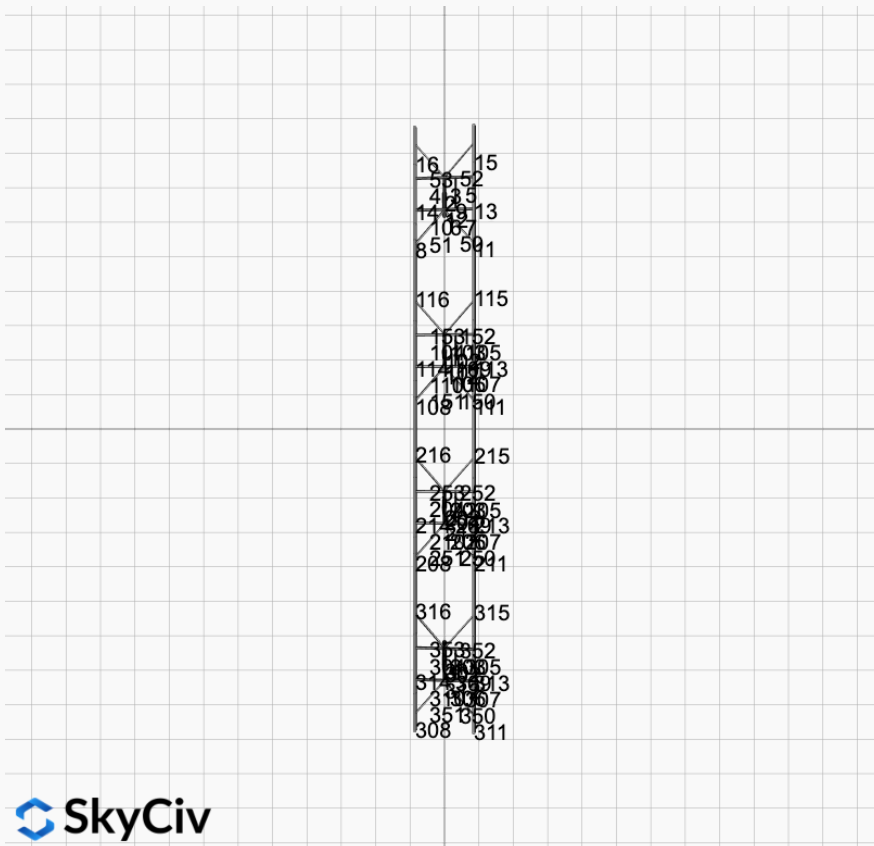
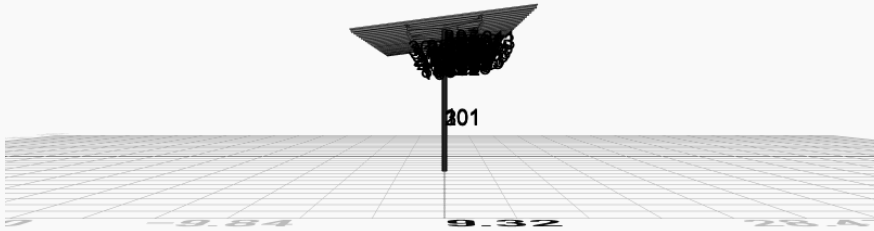
AutoDesigner Input

```
{"wind_speed_override":90,"snow_load_override":30,"direct_snow_load":false,"add_angle_brace":true,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"Columbus|Lozza 5x11 - V1Jb","site_address":"420 Columbus Ave, Valhalla, NY 10595, USA","module_width":44.6,"module_length":82.6,"number_rows":"5","number_columns":11,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":15,"tilt_angle":7.5,"ground_clearance":14,"risk_category":"I","exposure_category":"B","frame_duty_override":"XD","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":true}
```

Design Notes:

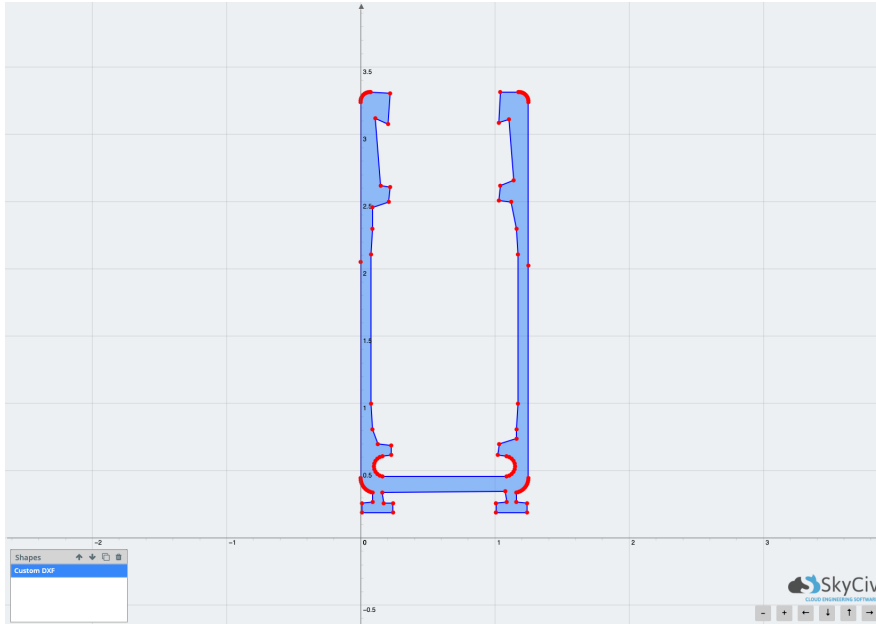
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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





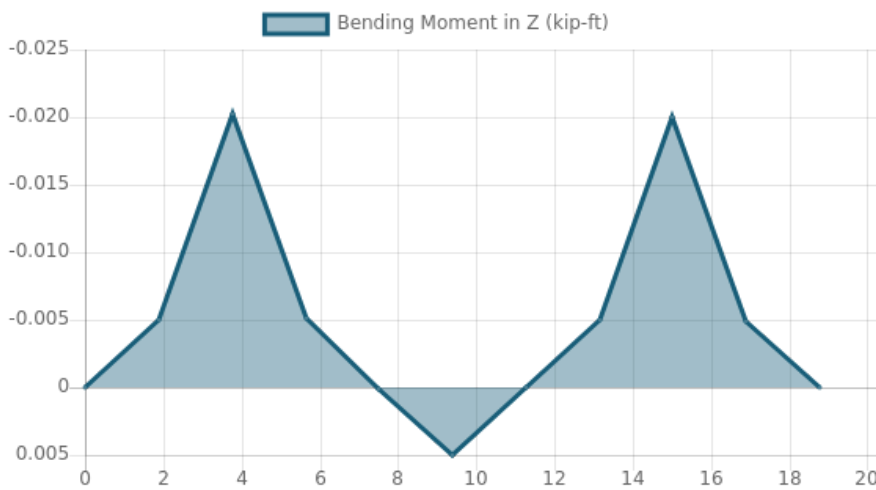
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.48333333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0627 kip/ft
Snow (Y): -0.0082 kip/ft
Wind uplift Case A: 0.0296 kip/ft
Wind uplift Case A: 0.0296 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0415 kip/ft

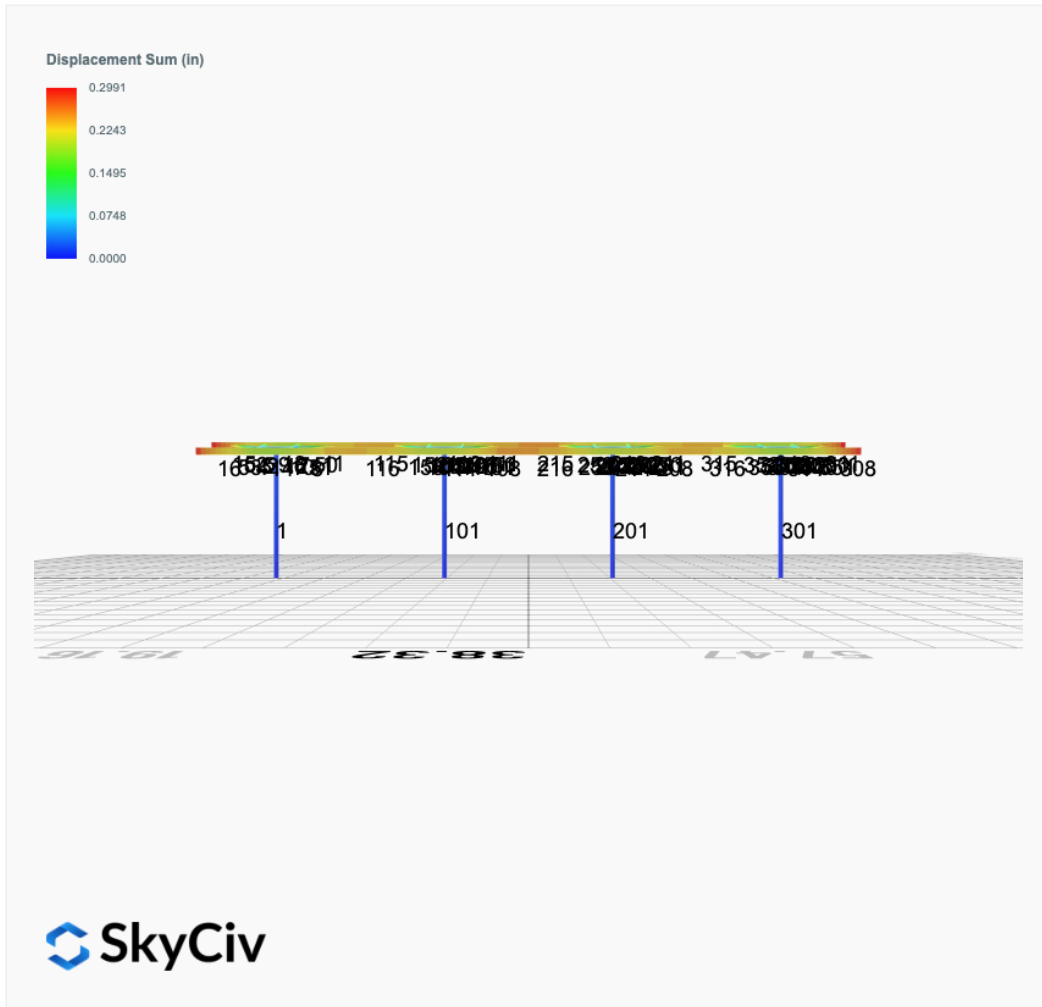


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	7.93155009	0.230	PASS
Material Yield	34.5	7.93155009	0.230	PASS
Material Strength	37	7.93155009	0.214	PASS

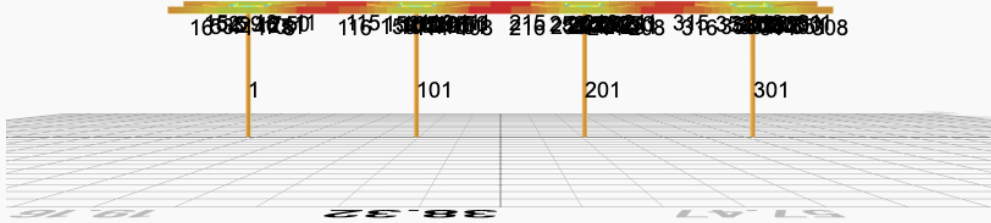
Member 1, ULS: 1. 1.4D



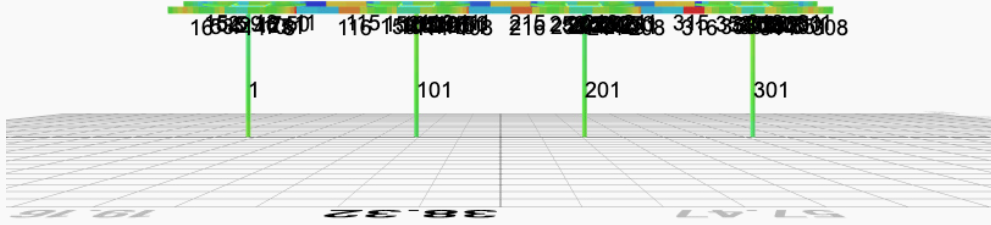
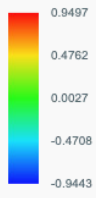
FEM Results (Envelope Worst Case for each member)



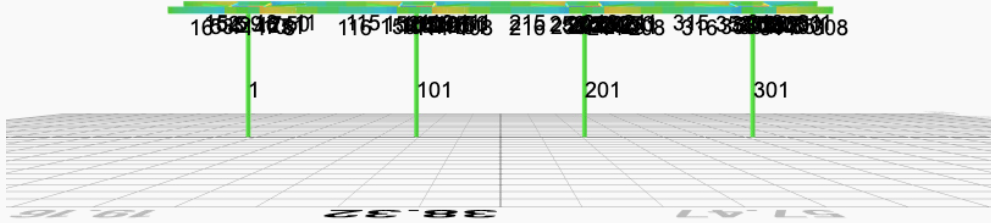
Top Bending Stress Z (ksi)



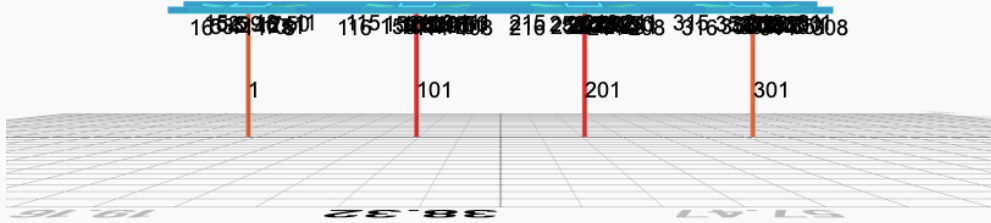
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0004	2.6356	-0.0037	-0.0181	0.0035	0.0384
ULS: 2. D + L	-0.0004	2.6356	-0.0037	-0.0181	0.0035	0.0384
ULS: 3. D + (S or Lr or R)	-0.0015	8.8886	-0.0129	-0.0641	0.0136	0.0662
ULS: 3. D + (S or Lr or R)	-0.0004	2.6356	-0.0037	-0.0181	0.0035	0.0384
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0013	7.3253	-0.0106	-0.0526	0.0111	0.0592
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0004	2.6356	-0.0037	-0.0181	0.0035	0.0384
ULS: 5b. D + 0.7E	-0.0004	2.6356	-0.0037	-0.0181	0.0035	0.0384
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0013	7.3253	-0.0106	-0.0526	0.0111	0.0592
ULS: 8. 0.6D + 0.7E	-0.0003	1.5814	-0.0022	-0.0109	0.0021	0.0230
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2913	4.7561	-0.0064	-0.0320	0.0031	6.8736
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2913	4.7561	-0.0064	-0.0320	0.0031	6.8736
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1876	1.2218	-0.0016	-0.0078	0.0029	-1.4548
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1750	1.3996	-0.0023	-0.0113	0.0046	-7.0823
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2194	8.9157	-0.0126	-0.0630	0.0108	5.1857
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2194	8.9157	-0.0126	-0.0630	0.0108	5.1857
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1398	6.2650	-0.0090	-0.0448	0.0106	-1.0607
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1303	6.3983	-0.0095	-0.0475	0.0119	-5.2812
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2186	4.2260	-0.0057	-0.0285	0.0032	5.1648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2186	4.2260	-0.0057	-0.0285	0.0032	5.1648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1406	1.5752	-0.0021	-0.0103	0.0031	-1.0815
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1312	1.7086	-0.0026	-0.0130	0.0043	-5.3021
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2911	3.7018	-0.0050	-0.0248	0.0017	6.8582
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2911	3.7018	-0.0050	-0.0248	0.0017	6.8582
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1878	0.1675	-0.0001	-0.0005	0.0015	-1.4702
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1752	0.3453	-0.0008	-0.0040	0.0032	-7.0976

Worst Case Reactions LFRD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

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

Result	Value (kip, kip-ft)
Axial	14.9345
Shear X	-0.4885
Shear Z	-0.0215
Moment X	-0.1086
Moment Y (Twist)	0.0218
Moment Z	13.3044

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

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

Result	Value (kip, kip-ft)
Axial	8.9157
Shear X	-0.2913
Shear Z	-0.0129
Moment X	-0.0641
Moment Y (Twist)	0.0136
Moment Z	7.0976

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	0.0004	2.7542	0.0021	0.0104	-0.0008	0.0287
ULS: 2. D + L	0.0004	2.7542	0.0021	0.0104	-0.0008	0.0287
ULS: 3. D + (S or Lr or R)	0.0015	9.3889	0.0081	0.0404	-0.0030	0.0302
ULS: 3. D + (S or Lr or R)	0.0004	2.7542	0.0021	0.0104	-0.0008	0.0287
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0013	7.7302	0.0066	0.0329	-0.0024	0.0298

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0004	2.7542	0.0021	0.0104	-0.0008	0.0287
ULS: 5b. D + 0.7E	0.0004	2.7542	0.0021	0.0104	-0.0008	0.0287
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0013	7.7302	0.0066	0.0329	-0.0024	0.0298
ULS: 8. 0.6D + 0.7E	0.0003	1.6525	0.0013	0.0062	-0.0005	0.0172
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2842	5.0050	0.0044	0.0221	-0.0030	6.8808
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2842	5.0050	0.0044	0.0221	-0.0030	6.8808
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1961	1.2538	0.0007	0.0034	0.0000	-1.5096
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1607	1.4403	0.0006	0.0029	0.0012	-7.0592
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2122	9.4183	0.0083	0.0416	-0.0041	5.1689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2122	9.4183	0.0083	0.0416	-0.0041	5.1689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1480	6.6050	0.0055	0.0276	-0.0019	-1.1239
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1214	6.7448	0.0054	0.0273	-0.0009	-5.2861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2131	4.4423	0.0038	0.0192	-0.0024	5.1678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2131	4.4423	0.0038	0.0192	-0.0024	5.1678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1472	1.6289	0.0010	0.0051	-0.0002	-1.1250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1206	1.7687	0.0010	0.0048	0.0007	-5.2872
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2844	3.9033	0.0036	0.0179	-0.0027	6.8694
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2844	3.9033	0.0036	0.0179	-0.0027	6.8694
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1959	0.1522	-0.0002	-0.0008	0.0003	-1.5211
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1605	0.3386	-0.0003	-0.0012	0.0016	-7.0706

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	15.7964
Shear X	-0.4744
Shear Z	0.0141
Moment X	0.0717
Moment Y (Twist)	0.0067
Moment Z	13.2705

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.4183
Shear X	-0.2844
Shear Z	0.0083
Moment X	0.0416
Moment Y (Twist)	0.0041
Moment Z	7.0706

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	0.0004	2.7542	-0.0021	-0.0104	0.0008	0.0287
ULS: 2. D + L	0.0004	2.7542	-0.0021	-0.0104	0.0008	0.0287
ULS: 3. D + (S or Lr or R)	0.0015	9.3889	-0.0081	-0.0404	0.0030	0.0302
ULS: 3. D + (S or Lr or R)	0.0004	2.7542	-0.0021	-0.0104	0.0008	0.0287
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0013	7.7302	-0.0066	-0.0329	0.0024	0.0298
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0004	2.7542	-0.0021	-0.0104	0.0008	0.0287
ULS: 5b. D + 0.7E	0.0004	2.7542	-0.0021	-0.0104	0.0008	0.0287
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0013	7.7302	-0.0066	-0.0329	0.0024	0.0298
ULS: 8. 0.6D + 0.7E	0.0003	1.6525	-0.0013	-0.0062	0.0005	0.0172
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2842	5.0050	-0.0044	-0.0221	0.0030	6.8808
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2842	5.0050	-0.0044	-0.0221	0.0030	6.8808
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1961	1.2538	-0.0007	-0.0034	-0.0000	-1.5096
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1607	1.4403	-0.0006	-0.0029	-0.0012	-7.0592

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2122	9.4183	-0.0083	-0.0416	0.0041	5.1689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2122	9.4183	-0.0083	-0.0416	0.0041	5.1689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1480	6.6050	-0.0055	-0.0276	0.0019	-1.1239
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1214	6.7448	-0.0054	-0.0273	0.0009	-5.2861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2131	4.4423	-0.0038	-0.0192	0.0024	5.1678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2131	4.4423	-0.0038	-0.0192	0.0024	5.1678
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1472	1.6289	-0.0010	-0.0051	0.0002	-1.1250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1206	1.7687	-0.0010	-0.0048	-0.0007	-5.2872
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2844	3.9033	-0.0036	-0.0179	0.0027	6.8694
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2844	3.9033	-0.0036	-0.0179	0.0027	6.8694
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1959	0.1522	0.0002	0.0008	-0.0003	-1.5211
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1605	0.3386	0.0003	0.0012	-0.0016	-7.0706

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	15.7963
Shear X	-0.4744
Shear Z	-0.0141
Moment X	-0.0717
Moment Y (Twist)	0.0067
Moment Z	13.2705

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.4183
Shear X	-0.2844
Shear Z	-0.0083
Moment X	-0.0416
Moment Y (Twist)	0.0041
Moment Z	7.0706

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	-0.0004	2.6356	0.0037	0.0181	-0.0035	0.0384
ULS: 2. D + L	-0.0004	2.6356	0.0037	0.0181	-0.0035	0.0384
ULS: 3. D + (S or Lr or R)	-0.0015	8.8886	0.0129	0.0641	-0.0136	0.0662
ULS: 3. D + (S or Lr or R)	-0.0004	2.6356	0.0037	0.0181	-0.0035	0.0384
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0013	7.3253	0.0106	0.0526	-0.0111	0.0592
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0004	2.6356	0.0037	0.0181	-0.0035	0.0384
ULS: 5b. D + 0.7E	-0.0004	2.6356	0.0037	0.0181	-0.0035	0.0384
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0013	7.3253	0.0106	0.0526	-0.0111	0.0592
ULS: 8. 0.6D + 0.7E	-0.0003	1.5814	0.0022	0.0109	-0.0021	0.0230
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2913	4.7561	0.0064	0.0320	-0.0031	6.8736
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2913	4.7561	0.0064	0.0320	-0.0031	6.8736
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1876	1.2218	0.0016	0.0078	-0.0029	-1.4548
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1750	1.3996	0.0023	0.0113	-0.0046	-7.0823
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2194	8.9157	0.0126	0.0630	-0.0108	5.1857
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2194	8.9157	0.0126	0.0630	-0.0108	5.1857
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1398	6.2650	0.0090	0.0448	-0.0106	-1.0607
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1303	6.3983	0.0095	0.0475	-0.0119	-5.2812
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2186	4.2260	0.0057	0.0285	-0.0032	5.1648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2186	4.2260	0.0057	0.0285	-0.0032	5.1648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.1406	1.5752	0.0021	0.0103	-0.0031	-1.0815
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1312	1.7086	0.0026	0.0130	-0.0043	-5.3021

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2911	3.7018	0.0050	0.0248	-0.0017	6.8582
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2911	3.7018	0.0050	0.0248	-0.0017	6.8582
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1878	0.1675	0.0001	0.0005	-0.0015	-1.4702
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1752	0.3453	0.0008	0.0040	-0.0032	-7.0976

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.9345
Shear X	-0.4885
Shear Z	0.0215
Moment X	0.1086
Moment Y (Twist)	0.0218
Moment Z	13.3041

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.9157
Shear X	-0.2913
Shear Z	0.0129
Moment X	0.0641
Moment Y (Twist)	0.0136
Moment Z	7.0976

Project Details

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

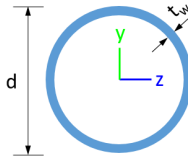


Design Input Information

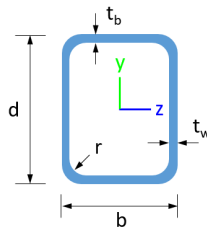
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

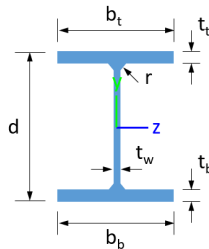
Section Dimensions



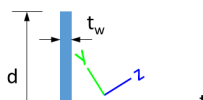
ID	Name	d (in)	t _w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30



10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	32.82	6.46	56.26	44.91
14	159.30	97.43	33.11	6.46	56.26	44.91
15	159.30	105.96	46.90	6.46	56.26	44.91
16	159.30	105.96	46.90	6.46	56.26	44.91
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15
101	251.16	43.18	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.18	6.46	56.26	44.91
114	159.30	97.43	31.65	6.46	56.26	44.91
115	159.30	32.87	21.63	6.46	56.26	44.91
116	159.30	32.87	21.65	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	251.16	43.18	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	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	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	31.19	6.46	56.26	44.91
214	159.30	97.43	31.65	6.46	56.26	44.91
215	159.30	32.87	19.44	6.46	56.26	44.91
216	159.30	32.87	21.41	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	251.16	43.18	42.30	42.30	75.35	75.35

301	251.10	248.88	27.16	27.16	75.30	75.30
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	105.96	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	105.96	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	32.83	6.46	56.26	44.91
314	159.30	97.43	33.11	6.46	56.26	44.91
315	159.30	32.87	21.58	6.46	56.26	44.91
316	159.30	32.87	22.08	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	0.346	0.315	0.005	0.006	0.000	0.501	#21	0.854	Not Required	Pass
2	0.001	0.474	0.023	0.097	0.003	0.497	#21	0.054	Not Required	Pass
3	0.001	0.688	0.029	0.069	0.012	0.717	#21	0.046	Not Required	Pass
4	0.001	0.644	0.009	0.065	0.001	0.654	#21	0.082	Not Required	Pass
5	0.001	0.426	0.006	0.068	0.002	0.432	#21	0.076	Not Required	Pass
6	0.001	0.682	0.032	0.068	0.013	0.713	#21	0.046	Not Required	Pass
7	0.001	0.423	0.004	0.068	0.001	0.427	#21	0.076	Not Required	Pass
8	0.001	0.042	0.022	0.041	0.002	0.061	#24	0.102	Not Required	Pass
9	0.005	0.075	0.012	0.001	0.000	0.087	#21	0.137	Not Required	Pass
10	0.001	0.637	0.015	0.064	0.002	0.653	#21	0.082	Not Required	Pass
11	0.001	0.046	0.022	0.044	0.003	0.061	#21	0.068	Not Required	Pass
12	0.001	0.466	0.022	0.096	0.003	0.489	#21	0.054	Not Required	Pass
13	0.001	0.269	0.013	0.056	0.002	0.282	#21	0.204	Not Required	Pass
14	0.003	0.258	0.013	0.053	0.002	0.271	#21	0.306	Not Required	Pass
15	0.001	0.095	0.017	0.033	0.002	0.099	#21	0.242	Not Required	Pass
16	0.002	0.089	0.017	0.031	0.002	0.091	#21	0.242	Not Required	Pass
50	0.055	0.010	0.006	0.001	0.001	0.069	#24	0.783	Not Required	Pass
51	0.011	0.009	0.010	0.001	0.001	0.027	#24	0.522	Not Required	Pass
52	0.037	0.010	0.006	0.001	0.001	0.051	#21	0.783	Not Required	Pass
53	0.007	0.009	0.010	0.001	0.001	0.023	#21	0.522	Not Required	Pass
101	0.366	0.314	0.003	0.006	0.000	0.520	#21	0.854	Not Required	Pass
102	0.001	0.491	0.022	0.102	0.003	0.514	#21	0.054	Not Required	Pass
103	0.001	0.723	0.032	0.072	0.013	0.755	#21	0.046	Not Required	Pass
104	0.001	0.677	0.013	0.068	0.002	0.691	#21	0.082	Not Required	Pass
105	0.001	0.448	0.006	0.072	0.002	0.454	#21	0.076	Not Required	Pass
106	0.001	0.728	0.033	0.073	0.013	0.761	#21	0.046	Not Required	Pass
107	0.001	0.451	0.005	0.072	0.001	0.457	#21	0.076	Not Required	Pass

108	0.001	0.055	0.017	0.040	0.002	0.072	#21	0.102	Not Required	Pass
109	0.006	0.074	0.011	0.001	0.000	0.088	#21	0.137	Not Required	Pass
110	0.001	0.682	0.012	0.068	0.002	0.694	#21	0.082	Not Required	Pass
111	0.001	0.058	0.017	0.043	0.002	0.076	#21	0.102	Not Required	Pass
112	0.001	0.497	0.023	0.103	0.003	0.520	#21	0.054	Not Required	Pass
113	0.001	0.221	0.011	0.055	0.002	0.231	#21	0.306	Not Required	Pass
114	0.003	0.213	0.012	0.052	0.002	0.220	#21	0.306	Not Required	Pass
115	0.004	0.246	0.031	0.042	0.004	0.262	#21	0.780	Not Required	Pass
116	0.006	0.231	0.031	0.040	0.004	0.246	#21	0.780	Not Required	Pass
150	0.051	0.010	0.006	0.001	0.001	0.065	#24	0.783	Not Required	Pass
151	0.010	0.009	0.010	0.001	0.001	0.026	#24	0.522	Not Required	Pass
152	0.052	0.010	0.006	0.001	0.001	0.066	#21	0.783	Not Required	Pass
153	0.010	0.009	0.010	0.001	0.001	0.026	#21	0.522	Not Required	Pass
201	0.366	0.314	0.003	0.006	0.000	0.520	#21	0.854	Not Required	Pass
202	0.001	0.497	0.023	0.103	0.003	0.520	#21	0.054	Not Required	Pass
203	0.001	0.728	0.033	0.073	0.013	0.761	#21	0.046	Not Required	Pass
204	0.001	0.682	0.012	0.068	0.002	0.694	#21	0.082	Not Required	Pass
205	0.001	0.451	0.005	0.072	0.001	0.457	#21	0.076	Not Required	Pass
206	0.001	0.723	0.032	0.072	0.013	0.755	#21	0.046	Not Required	Pass
207	0.001	0.448	0.006	0.072	0.002	0.454	#21	0.076	Not Required	Pass
208	0.001	0.050	0.017	0.040	0.002	0.068	#21	0.102	Not Required	Pass
209	0.006	0.074	0.011	0.001	0.000	0.088	#21	0.137	Not Required	Pass
210	0.001	0.677	0.013	0.068	0.002	0.691	#21	0.082	Not Required	Pass
211	0.001	0.054	0.018	0.042	0.002	0.072	#21	0.102	Not Required	Pass
212	0.001	0.491	0.022	0.102	0.003	0.514	#21	0.054	Not Required	Pass
213	0.001	0.221	0.011	0.055	0.002	0.231	#21	0.306	Not Required	Pass
214	0.003	0.213	0.012	0.052	0.002	0.220	#21	0.306	Not Required	Pass
215	0.005	0.265	0.025	0.043	0.004	0.281	#21	0.780	Not Required	Pass
216	0.006	0.249	0.024	0.040	0.004	0.265	#21	0.780	Not Required	Pass
250	0.052	0.010	0.006	0.001	0.001	0.066	#21	0.783	Not Required	Pass
251	0.010	0.009	0.010	0.001	0.001	0.026	#21	0.522	Not Required	Pass
252	0.051	0.010	0.006	0.001	0.001	0.065	#24	0.783	Not Required	Pass
253	0.010	0.009	0.010	0.001	0.001	0.026	#24	0.522	Not Required	Pass
301	0.346	0.315	0.005	0.006	0.000	0.501	#21	0.854	Not Required	Pass
302	0.001	0.466	0.022	0.096	0.003	0.489	#21	0.054	Not Required	Pass
303	0.001	0.682	0.032	0.068	0.013	0.713	#21	0.046	Not Required	Pass
304	0.001	0.637	0.015	0.064	0.002	0.653	#21	0.082	Not Required	Pass
305	0.001	0.423	0.004	0.068	0.001	0.427	#21	0.076	Not Required	Pass
306	0.001	0.688	0.029	0.069	0.012	0.717	#21	0.046	Not Required	Pass
307	0.001	0.426	0.006	0.068	0.002	0.432	#21	0.076	Not Required	Pass
308	0.002	0.089	0.017	0.031	0.002	0.091	#21	0.363	Not Required	Pass
309	0.005	0.075	0.012	0.001	0.000	0.087	#21	0.137	Not Required	Pass
310	0.001	0.644	0.009	0.065	0.001	0.654	#21	0.082	Not Required	Pass
311	0.001	0.095	0.017	0.033	0.002	0.099	#21	0.242	Not Required	Pass
312	0.001	0.474	0.023	0.097	0.003	0.497	#21	0.054	Not Required	Pass
313	0.001	0.269	0.013	0.056	0.002	0.282	#21	0.204	Not Required	Pass
314	0.003	0.258	0.013	0.053	0.002	0.271	#21	0.306	Not Required	Pass
315	0.004	0.241	0.031	0.044	0.004	0.257	#21	0.520	Not Required	Pass
316	0.006	0.227	0.031	0.041	0.004	0.242	#21	0.780	Not Required	Pass
350	0.037	0.010	0.006	0.001	0.001	0.051	#21	0.783	Not Required	Pass
351	0.007	0.009	0.010	0.001	0.001	0.023	#21	0.522	Not Required	Pass
352	0.055	0.010	0.006	0.001	0.001	0.069	#24	0.783	Not Required	Pass

353	0.011	0.009	0.010	0.001	0.001	0.027	#24	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
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
------------	--------------	---------

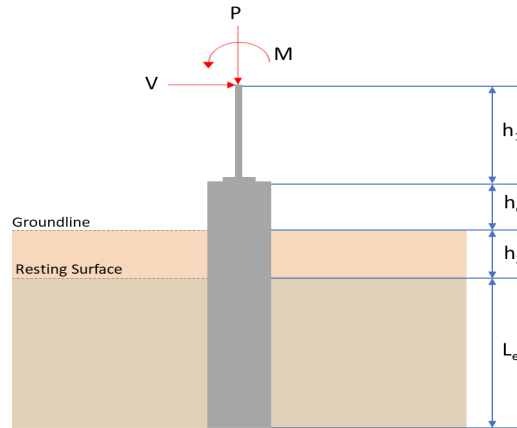
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 4.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.916	14.934
V_x (kip)	-0.291	-0.488
V_z (kip)	-0.013	-0.022
M_x (kipft)	-0.064	-0.109
M_z (kipft)	7.098	13.304

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.291 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.098 \text{ kipft}) + ((-0.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.1303 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.282 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0020701 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010191 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.88988 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.282 \text{ ft}), (0.88988 \text{ ft})]$$

$$L_{e,req} = 4.282 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.282 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.95156$$

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

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

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

$$q = \frac{(8.916 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55725 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55725 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27863$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.046338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.1303 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.1303 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.1303 \text{ kipft/ft})) + (4 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0411 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.1303 \text{ kipft/ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.1303 \text{ kipft/ft})) + (2 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.189 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.1303 \text{ kipft/ft})) + ((-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.608 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0411 \text{ ft})}{2}$$

$$p_a = 0.22808 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.189 \text{ kip/ft}^2)}{(0.22808 \text{ kip/ft}^2)}$$

$$Ratio = 0.82864$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.608 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.90074$$

Status: **PASS**
Ratio: **0.830**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.0020701 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010191 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010191 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.010191 \text{ kipft/ft})) + (4 \times (-0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.142 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010191 \text{ kipft/ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.010191 \text{ kipft/ft})) + (2 \times (-0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.00050957 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010191 \text{ kipft/ft})) + ((-0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0032791 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.142 \text{ ft})}{2}$$

$$p_a = 0.23565 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00050957 \text{ kip/ft}^2)}{(0.23565 \text{ kip/ft}^2)}$$

$$Ratio = 0.0021624$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

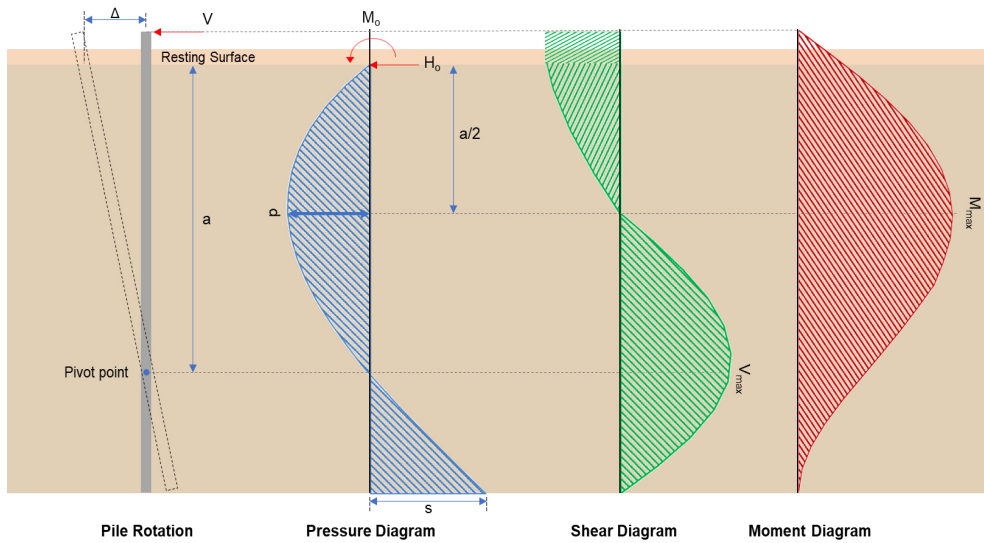
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0032791 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.0048579$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.488 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.077707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.304 \text{ kipft}) + ((-0.488 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1185 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1185 \text{ kipft/ft})}{(-0.077707 \text{ kip/ft})}$$

$$E = 27.262 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1185 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.077707 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (2.1185 \text{ kipft/ft})) + (4 \times (-0.077707 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.077707 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.1185 \text{ kipft/ft})) + (4 \times (-0.077707 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0372 \text{ ft}$$

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

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

$$V_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0372 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0372 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5451 \text{ kip}$$

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

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

$$M_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(27.262 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 7.848 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.022 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0035032 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.109 \text{ kipft}) + ((-0.022 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017357 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017357 \text{ kipft/ft})}{(-0.0035032 \text{ kip/ft})}$$

$$E = 4.9545 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017357 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0035032 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.017357 \text{ kipft/ft})) + (4 \times (-0.0035032 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1414 \text{ ft}$$

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

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

$$V_{max} = ((-0.0035032 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1414 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1414 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.036549 \text{ kip}$$

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

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

$$M_{max} = ((-0.0035032 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.9545 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.076546 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.934 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.1 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.1 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(14.934 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0055824$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.934 \text{ kip} \rightarrow 14934 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(14934 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.48 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.48 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.48 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.39 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.5451 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(3.5451 \text{ kip})}{(111.39 \text{ kip})}$$

$$Ratio = 0.031826$$

Status: **PASS**
Ratio: **0.030**

Considering z-direction:

$V_{max} = 0.036549 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.036549 \text{ kip})}{(111.39 \text{ kip})}$$

$$Ratio = 0.00032811$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 7.848 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.848 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.031442$$

Status: **PASS**
Ratio: **0.030**

Considering z-direction:

$M_{max} = 0.076546 \text{ kipft}$ - Maximum moment in the z-direction,

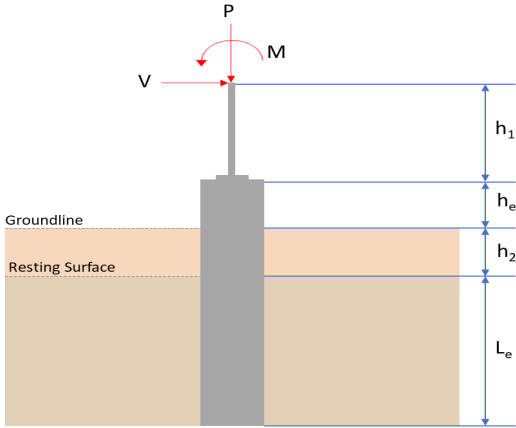
Ratio - Capacity

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

$$Ratio = \frac{(0.076546 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00030667$$

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

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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <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="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.418</td> <td>15.796</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.284</td> <td>-0.474</td> </tr> <tr> <td>V_z (kip)</td> <td>0.008</td> <td>0.014</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.042</td> <td>0.072</td> </tr> <tr> <td>M_z (kipft)</td> <td>7.071</td> <td>13.270</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	9.418	15.796	V_x (kip)	-0.284	-0.474	V_z (kip)	0.008	0.014	M_x (kipft)	0.042	0.072	M_z (kipft)	7.071	13.270	
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)	9.418	15.796																										
V_x (kip)	-0.284	-0.474																										
V_z (kip)	0.008	0.014																										
M_x (kipft)	0.042	0.072																										
M_z (kipft)	7.071	13.270																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.284 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.045223 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.071 \text{ kipft}) + ((-0.284 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.126 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2811 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.042 \text{ kipft}) + ((0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0066879 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.84317 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.2811 \text{ ft}), (0.84317 \text{ ft})]$$

$$L_{e,req} = 4.281 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.281 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.95133$$

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

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

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

$$q = \frac{(9.418 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58862 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58862 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29431$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.045223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.126 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.126 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.126 \text{ kipft/ft})) + (4 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0403 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.126 \text{ kipft/ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.126 \text{ kipft/ft})) + (2 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.18897 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.126 \text{ kipft/ft})) + ((-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.60694 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0403 \text{ ft})}{2}$$

$$p_a = 0.22802 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18897 \text{ kip/ft}^2)}{(0.22802 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82873$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.60694 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89916$$

Status: **PASS**
Ratio: **0.830**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.0012739 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0066879 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0066879 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.0066879 \text{ kipft/ft})) + (4 \times (0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1364 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0066879 \text{ kipft/ft})) + (3 \times (0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (0.0066879 \text{ kipft/ft})) + (2 \times (0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.002269 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0066879 \text{ kipft/ft})) + ((0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0056617 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.1364 \text{ ft})}{2}$$

$$p_a = 0.23523 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.002269 \text{ kip/ft}^2)}{(0.23523 \text{ kip/ft}^2)}$$

$$Ratio = 0.0096459$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

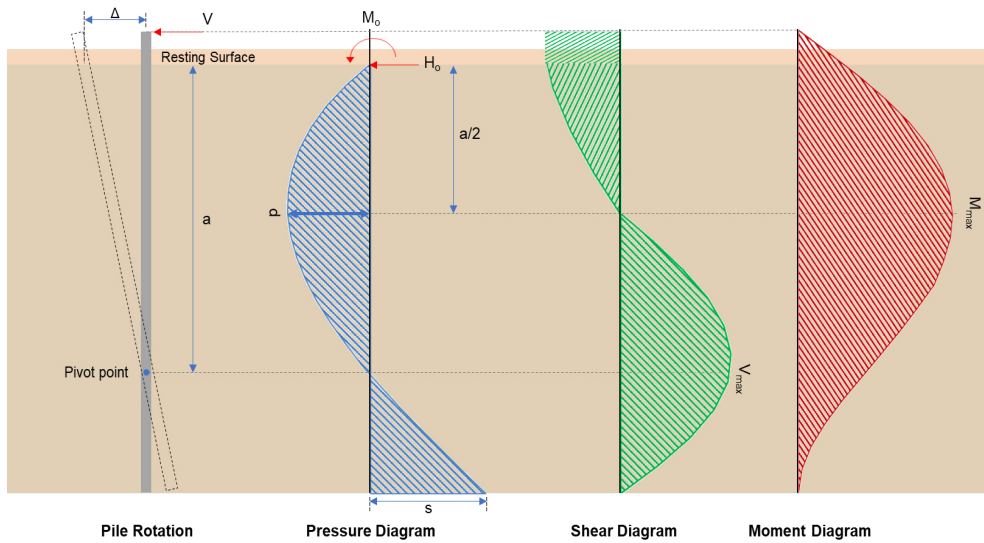
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0056617 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.0083877$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.474 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.075478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.27 \text{ kipft}) + ((-0.474 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1131 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1131 \text{ kipft/ft})}{(-0.075478 \text{ kip/ft})}$$

$$E = 27.996 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1131 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (2.1131 \text{ kipft/ft})) + (4 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.075478 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.1131 \text{ kipft/ft})) + (4 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0363 \text{ ft}$$

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

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

$$V_{max} = ((-0.075478 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (27.996 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0363 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (27.996 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0363 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5309 \text{ kip}$$

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

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

$$M_{max} = ((-0.075478 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(27.996 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (27.996 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (27.996 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 7.8196 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.072 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011465 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.011465 \text{ kipft/ft})}{(0.0022293 \text{ kip/ft})}$$

$$E = 5.1429 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011465 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.011465 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1382 \text{ ft}$$

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

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

$$V_{max} = ((0.0022293 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1382 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1382 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023917 \text{ kip}$$

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

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

$$M_{max} = ((0.0022293 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(5.1429 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050192 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.796 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.071 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.071 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(15.796 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0059046$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.796 \text{ kip} \rightarrow 15796 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15796 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.59 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.59 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.59 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.59 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.47 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.5309 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(3.5309 \text{ kip})}{(111.47 \text{ kip})}$$

$$Ratio = 0.031677$$

Considering z-direction:

$V_{max} = 0.023917 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.023917 \text{ kip})}{(111.47 \text{ kip})}$$

$$Ratio = 0.00021457$$

Status: **PASS**
Ratio: **0.030**

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 7.8196 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.8196 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.031328$$

Status: **PASS**
Ratio: **0.030**

Considering z-direction:

$M_{max} = 0.050192 \text{ kipft}$ - Maximum moment in the z-direction,

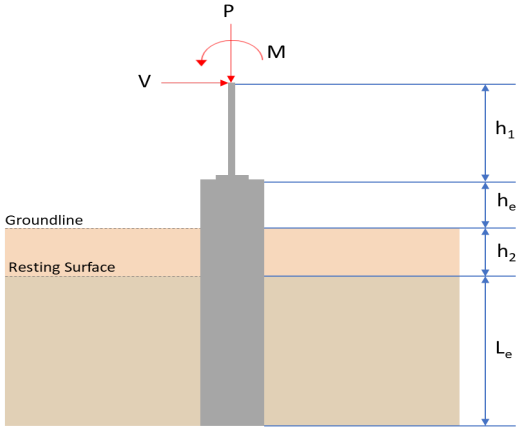
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.050192 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00020109$$

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

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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1084 1225 1184"> <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="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.418</td> <td>15.796</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.284</td> <td>-0.474</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.008</td> <td>-0.014</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.042</td> <td>-0.072</td> </tr> <tr> <td>M_z (kipft)</td> <td>7.071</td> <td>13.271</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	9.418	15.796	V_x (kip)	-0.284	-0.474	V_z (kip)	-0.008	-0.014	M_x (kipft)	-0.042	-0.072	M_z (kipft)	7.071	13.271	
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)	9.418	15.796																										
V_x (kip)	-0.284	-0.474																										
V_z (kip)	-0.008	-0.014																										
M_x (kipft)	-0.042	-0.072																										
M_z (kipft)	7.071	13.271																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.284 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.045223 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.071 \text{ kipft}) + ((-0.284 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.126 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2811 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.042 \text{ kipft}) + ((-0.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0066879 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.78064 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(4.2811 \text{ ft}), (0.78064 \text{ ft})]$$

$$L_{e,req} = 4.281 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(4.281 \text{ ft})}{(4.5 \text{ ft})}$$

$$Ratio = 0.95133$$

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

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

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

$$q = \frac{(9.418 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58862 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58862 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29431$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.045223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.126 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.126 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.126 \text{ kipft/ft})) + (4 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0403 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.126 \text{ kipft/ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.126 \text{ kipft/ft})) + (2 \times (-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.18897 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.126 \text{ kipft/ft})) + ((-0.045223 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.60694 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0403 \text{ ft})}{2}$$

$$p_a = 0.22802 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.18897 \text{ kip/ft}^2)}{(0.22802 \text{ kip/ft}^2)}$$

$$Ratio = 0.82873$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.60694 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.89916$$

Status: **PASS**
Ratio: **0.830**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.0012739 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0066879 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0066879 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.0066879 \text{ kipft/ft})) + (4 \times (-0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1364 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0066879 \text{ kipft/ft})) + (3 \times (-0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (0.0066879 \text{ kipft/ft})) + (2 \times (-0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.00039317 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0066879 \text{ kipft/ft})) + ((-0.0012739 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0022647 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.1364 \text{ ft})}{2}$$

$$p_a = 0.23523 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00039317 \text{ kip/ft}^2)}{(0.23523 \text{ kip/ft}^2)}$$

$$Ratio = 0.0016715$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

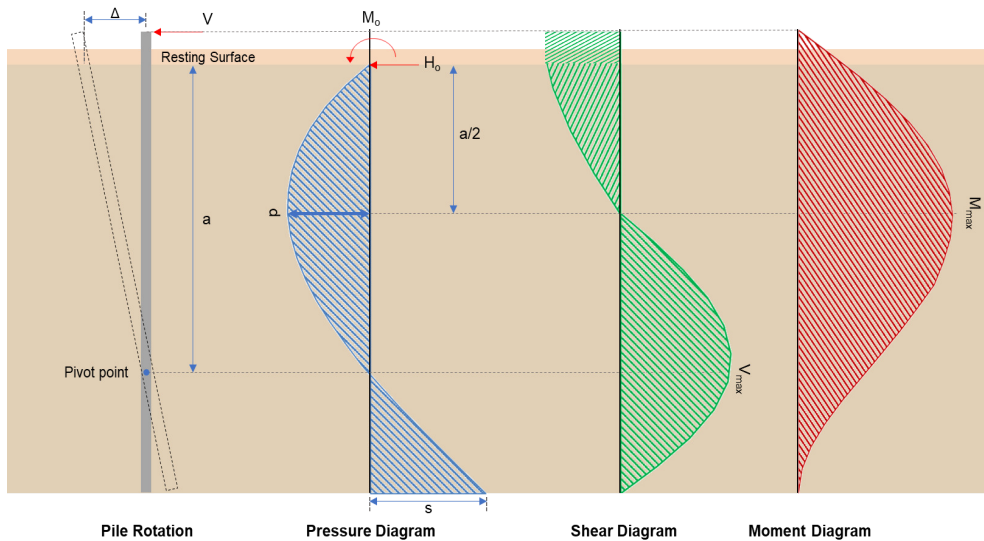
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0022647 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.0033551$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.474 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.075478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.271 \text{ kipft}) + ((-0.474 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1132 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1132 \text{ kipft/ft})}{(-0.075478 \text{ kip/ft})}$$

$$E = 27.998 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1132 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (2.1132 \text{ kipft/ft})) + (4 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.075478 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.1132 \text{ kip/ft})) + (4 \times (-0.075478 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0363 \text{ ft}$$

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

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

$$V_{max} = ((-0.075478 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (27.998 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0363 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (27.998 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0363 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5311 \text{ kip}$$

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

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

$$M_{max} = ((-0.075478 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(27.998 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (27.998 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (27.998 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0363 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 7.8201 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.072 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011465 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.011465 \text{ kipft/ft})}{(-0.0022293 \text{ kip/ft})}$$

$$E = 5.1429 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011465 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.011465 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1382 \text{ ft}$$

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

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

$$V_{max} = ((-0.0022293 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1382 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1382 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023917 \text{ kip}$$

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

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

$$M_{max} = ((-0.0022293 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(5.1429 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1429 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1382 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050192 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.796 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.071 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.071 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(15.796 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0059046$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.796 \text{ kip} \rightarrow 15796 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15796 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.59 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.59 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.59 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.59 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.47 \text{ kip}$$

Considering x-direction:

$V_{max} = 3.5311 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.5311 \text{ kip})}{(111.47 \text{ kip})}$$

$$Ratio = 0.031679$$

Considering z-direction:

$V_{max} = 0.023917 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.023917 \text{ kip})}{(111.47 \text{ kip})}$$

$$Ratio = 0.00021457$$

Status: **PASS**
Ratio: **0.030**

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 7.8201 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.8201 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.031331$$

Status: **PASS**
Ratio: **0.030**

Considering z-direction:

$M_{max} = 0.050192 \text{ kipft}$ - Maximum moment in the z-direction,

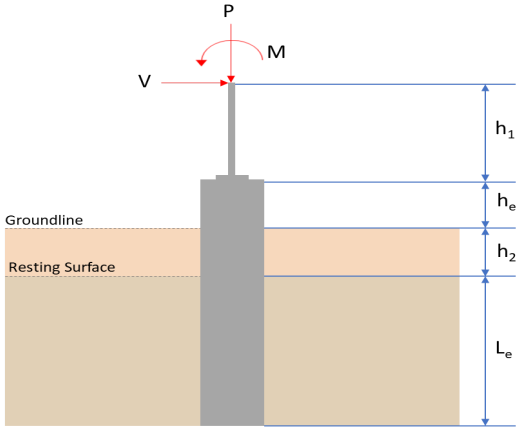
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.050192 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00020109$$

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

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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <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="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.916</td> <td>14.935</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.291</td> <td>-0.488</td> </tr> <tr> <td>V_z (kip)</td> <td>0.013</td> <td>0.022</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.064</td> <td>0.109</td> </tr> <tr> <td>M_z (kipft)</td> <td>7.098</td> <td>13.304</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	8.916	14.935	V_x (kip)	-0.291	-0.488	V_z (kip)	0.013	0.022	M_x (kipft)	0.064	0.109	M_z (kipft)	7.098	13.304	
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)	8.916	14.935																										
V_x (kip)	-0.291	-0.488																										
V_z (kip)	0.013	0.022																										
M_x (kipft)	0.064	0.109																										
M_z (kipft)	7.098	13.304																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.291 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.046338 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(7.098 \text{ kipft}) + ((-0.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.1303 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.282 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0020701 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.064 \text{ kipft}) + ((0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010191 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.97845 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.282 \text{ ft}), (0.97845 \text{ ft})]$$

$$L_{e,req} = 4.282 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.282 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.95156$$

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

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

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

$$q = \frac{(8.916 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55725 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55725 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27863$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.046338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.1303 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.1303 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.1303 \text{ kipft/ft})) + (4 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0411 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.1303 \text{ kipft/ft})) + (3 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.1303 \text{ kipft/ft})) + (2 \times (-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.189 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.1303 \text{ kipft/ft})) + ((-0.046338 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.608 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0411 \text{ ft})}{2}$$

$$p_a = 0.22808 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.189 \text{ kip/ft}^2)}{(0.22808 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82864$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.608 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90074$$

Status: **PASS**
Ratio: **0.830**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.0020701 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010191 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010191 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.010191 \text{ kipft/ft})) + (4 \times (0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.142 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010191 \text{ kipft/ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.010191 \text{ kipft/ft})) + (2 \times (0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.0035537 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010191 \text{ kipft/ft})) + ((0.0020701 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0087992 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.142 \text{ ft})}{2}$$

$$p_a = 0.23565 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0035537 \text{ kip/ft}^2)}{(0.23565 \text{ kip/ft}^2)}$$

$$Ratio = 0.01508$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

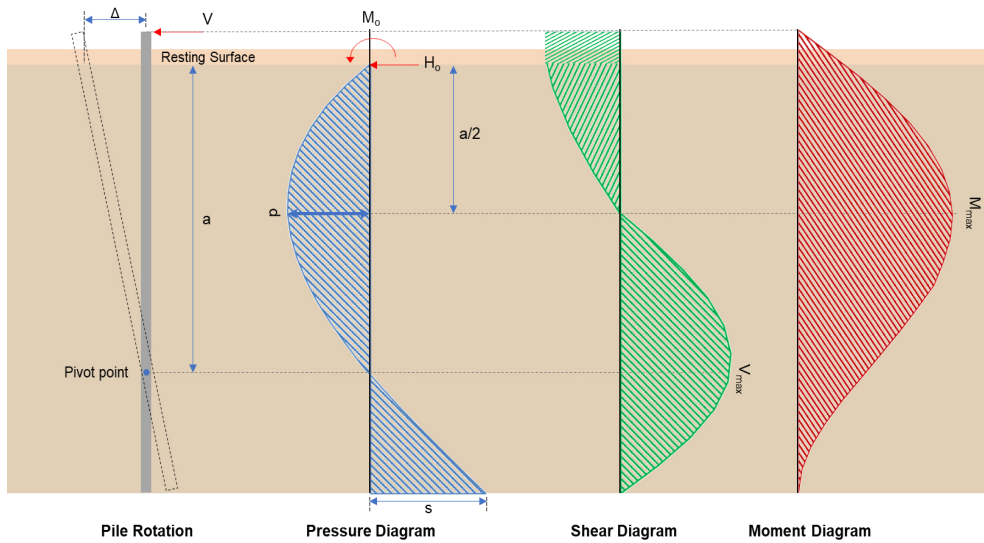
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0087992 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.013036$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.488 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.077707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(13.304 \text{ kipft}) + ((-0.488 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1185 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.1185 \text{ kipft/ft})}{(-0.077707 \text{ kip/ft})}$$

$$E = 27.262 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.1185 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.077707 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times 2.1185) + (4 \times (-0.077707) \times 4.5)}$$

$$a = \frac{(-0.077707 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.1185 \text{ kipft/ft})) + (4 \times (-0.077707 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0372 \text{ ft}$$

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

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

$$V_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0372 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0372 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5451 \text{ kip}$$

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

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

$$M_{max} = ((-0.077707 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(27.262 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (27.262 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0372 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 7.848 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.022 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0035032 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.109 \text{ kipft}) + ((0.022 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.017357 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.017357 \text{ kipft/ft})}{(0.0035032 \text{ kip/ft})}$$

$$E = 4.9545 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.017357 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0035032 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.017357 \text{ kipft/ft})) + (4 \times (0.0035032 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1414 \text{ ft}$$

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

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

$$V_{max} = ((0.0035032 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1414 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1414 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.036549 \text{ kip}$$

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

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

$$M_{max} = ((0.0035032 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.9545 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.9545 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1414 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.076546 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.935 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.1 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.1 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(14.935 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0055828$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.935 \text{ kip} \rightarrow 14935 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(14935 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.48 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.48 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.48 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.39 \text{ kip}$$

Considering x-direction:

V_{max} = 3.5451 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.5451 \text{ kip})}{(111.39 \text{ kip})}$$

$$Ratio = 0.031826$$

Considering z-direction:

$V_{max} = 0.036549 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.036549 \text{ kip})}{(111.39 \text{ kip})}$$

$$Ratio = 0.00032811$$

Status: **PASS**
Ratio: **0.030**

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 7.848 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.848 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.031442$$

Status: **PASS**
Ratio: **0.030**

Considering z-direction:

$M_{max} = 0.076546 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.076546 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00030667$$

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