

# Your Project Calculations



Project Name: MTSOLAR\_491C2J7BIHKK

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=MTSOLAR\\_491C2J7BIHKK&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/2\\_2024](https://platform.skyciv.com/structural?preload_name=MTSOLAR_491C2J7BIHKK&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/2_2024)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=3d3MU00TnZve3FEWri6rFywdkPiw5DZu6ote2ftOSIOJpzBONnxmYLz8P2fMGIOJ](https://platform.skyciv.com/structural-viewer?project_id=3d3MU00TnZve3FEWri6rFywdkPiw5DZu6ote2ftOSIOJpzBONnxmYLz8P2fMGIOJ)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	4P-19.75-6TOP-SD-45-L-5Hx14W-G6CK
<b>Duty Classification:</b>	SD
<b>Module Width:</b>	41.00 in
<b>Module Length:</b>	63.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	14
<b>Total Number of Modules:</b>	70
<b>Desired Tilt Angle:</b>	0
<b>Front Edge Clearance:</b>	12
<b>Total Array Height at Tilt:</b>	12.00 ft
<b>Total Frame Length:</b>	74.25 ft
<b>Frame Weight:</b>	4206 lbs
<b>Array Dimensions N/S:</b>	17.29 ft
<b>Array Dimensions E/W:</b>	74.67 ft
<b>Rail Length:</b>	207.50 in
<b>Rail Spacing:</b>	2.63 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	12.00 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.25 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.25 ft
<b>Foundation Volume:</b>	6.545 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.016765 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	201 Parkway W, Duncan, SC 29334, USA
<b>Wind Speed:</b>	101 mph
<b>Snow Load:</b>	10 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.006048 ksf



### Design Disclaimer

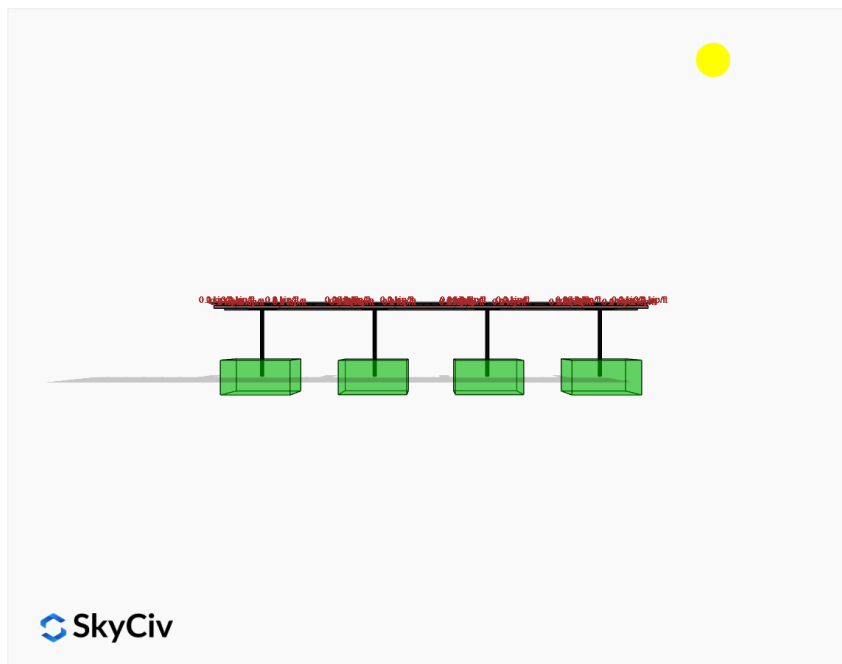
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

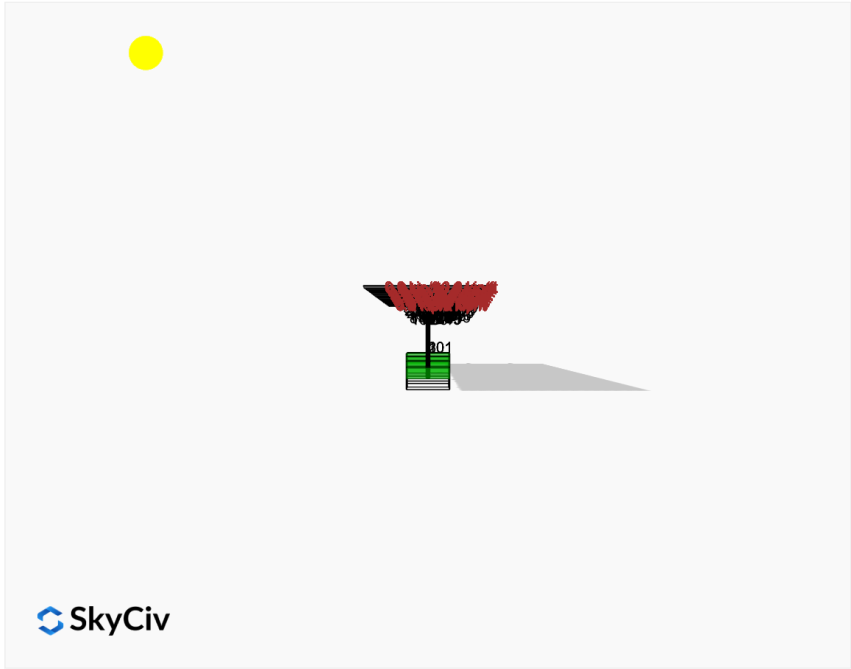
### AutoDesigner Input

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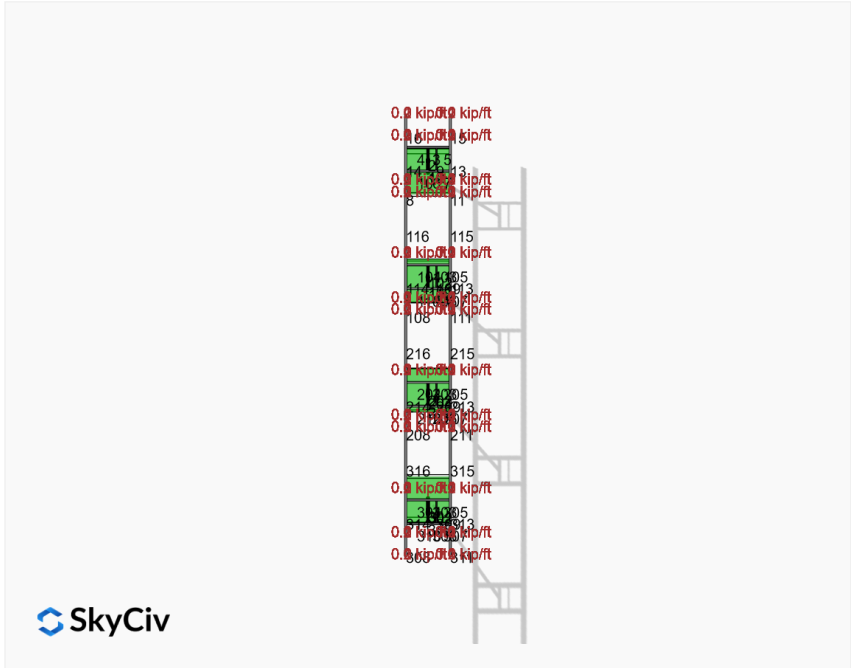
### Design Notes:

- 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)
- Foundation Design and Sizing is approximate only

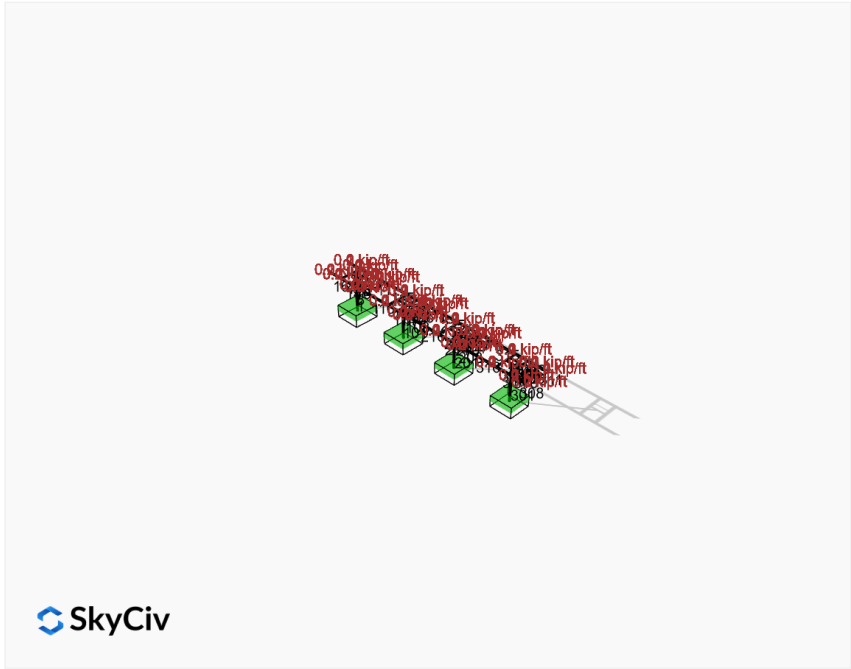




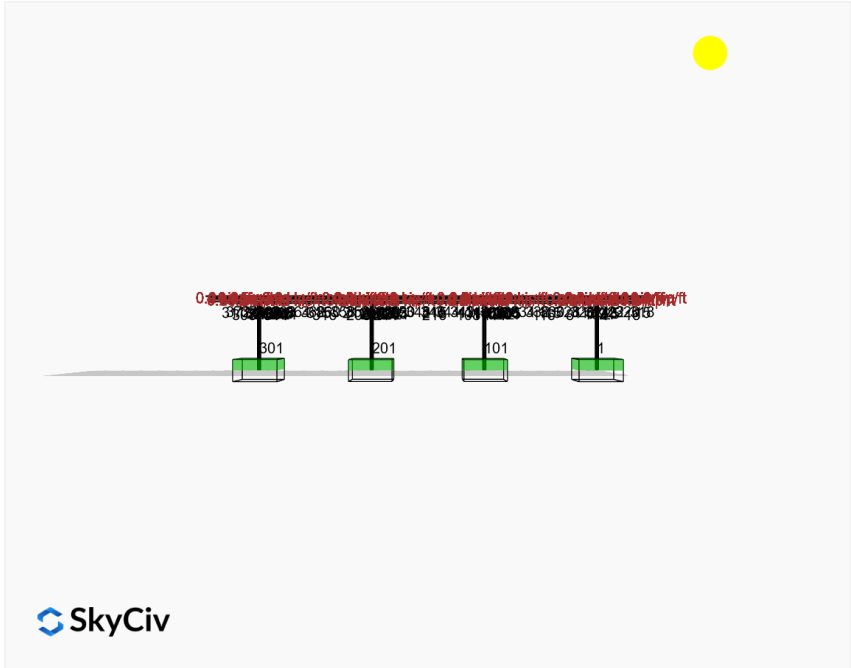
SkyCiv



SkyCiv

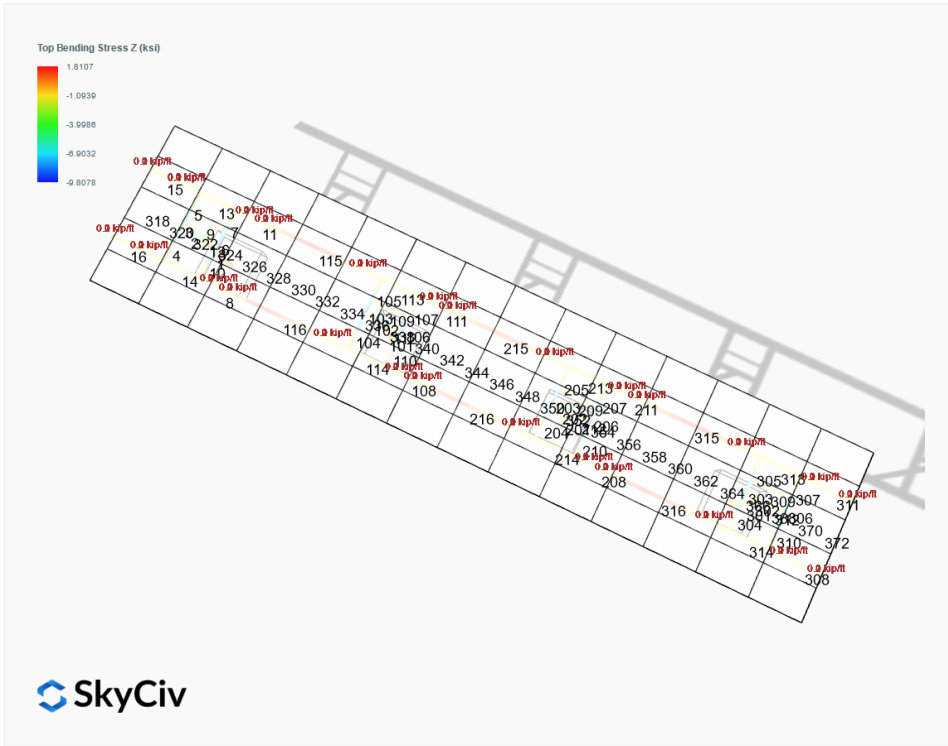
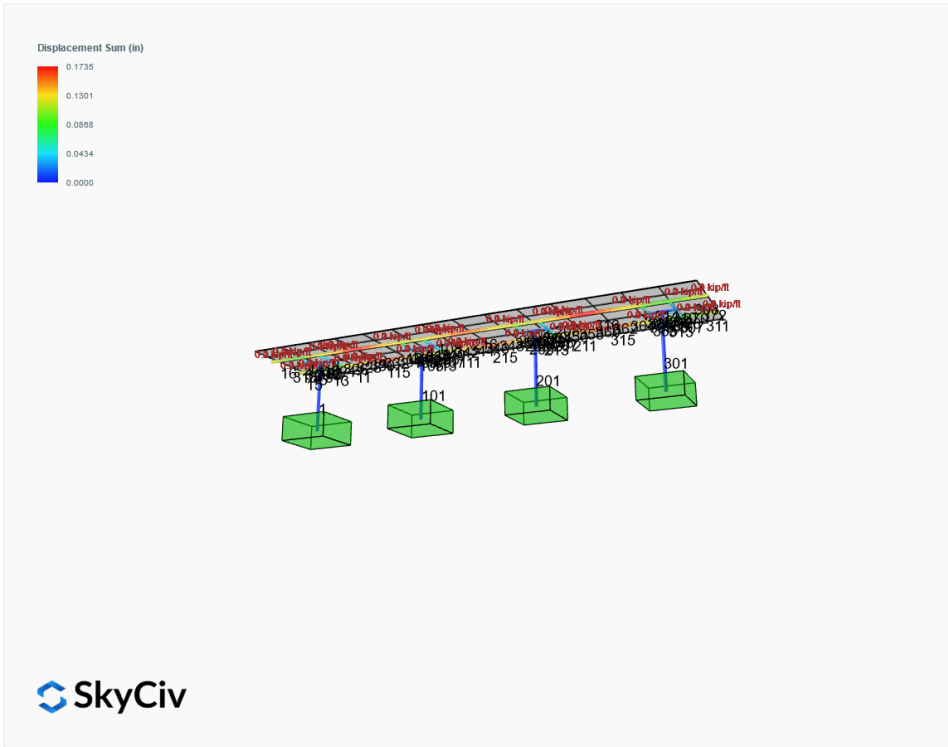


SkyCiv

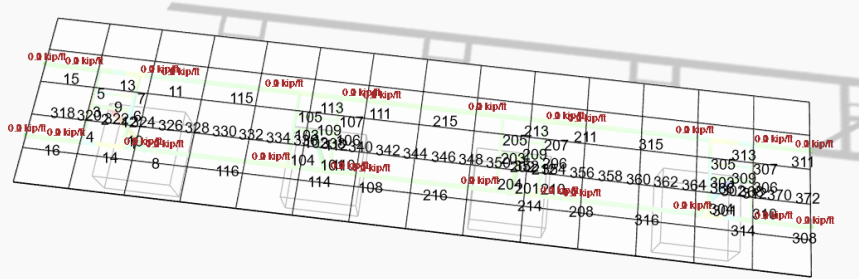


SkyCiv

# FEM Results (Envelope Worst Case for each member)

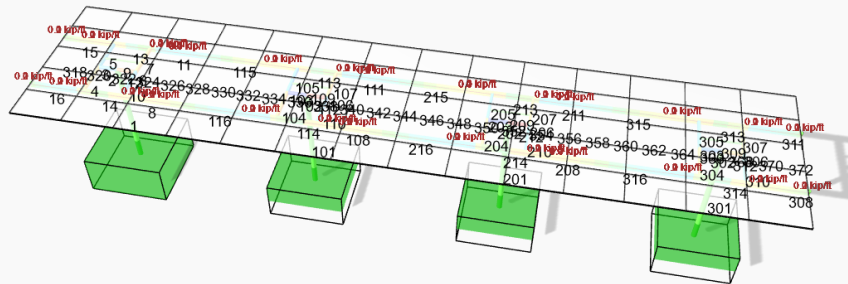


Top Bending Stress Y (ksi)



SkyCiv

Shear Stress Y (ksi)



SkyCiv



## 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	0.0000	2.0762	0.0297	0.1120	-0.0010	0.0206
ULS: 2. D + L	0.0000	2.0762	0.0297	0.1120	-0.0010	0.0206
ULS: 3. D + (S or Lr or R)	0.0001	3.8702	0.0612	0.2308	-0.0021	0.0204
ULS: 3. D + (S or Lr or R)	0.0000	2.0762	0.0297	0.1120	-0.0010	0.0206
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.4217	0.0533	0.2011	-0.0018	0.0205
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0762	0.0297	0.1120	-0.0010	0.0206
ULS: 5b. D + 0.7E	0.0000	2.0762	0.0297	0.1120	-0.0010	0.0206
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.4217	0.0533	0.2011	-0.0018	0.0205
ULS: 8. 0.6D + 0.7E	0.0000	1.2457	0.0178	0.0672	-0.0006	0.0124
ULS: 5a. D + 0.6W_Wind downforce Case A only	0.0131	4.1629	0.0659	0.2496	0.0050	-5.1352
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0131	4.1629	0.0659	0.2496	0.0050	-5.1352
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0130	4.1635	0.0669	0.2519	-0.0095	5.1760
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0128	0.4065	0.0001	0.0011	0.0071	-5.2482
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0099	4.9867	0.0805	0.3042	0.0027	-3.8464
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0099	4.9867	0.0805	0.3042	0.0027	-3.8464
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0097	4.9871	0.0812	0.3060	-0.0082	3.8870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0096	2.1694	0.0311	0.1179	0.0043	-3.9312
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0098	3.6412	0.0569	0.2152	0.0035	-3.8463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0098	3.6412	0.0569	0.2152	0.0035	-3.8463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0097	3.6417	0.0576	0.2170	-0.0074	3.8871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0096	0.8239	0.0075	0.0289	0.0051	-3.9310
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	0.0131	3.3324	0.0541	0.2048	0.0054	-5.1435
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0131	3.3324	0.0541	0.2048	0.0054	-5.1435
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0130	3.3330	0.0550	0.2071	-0.0091	5.1677
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0128	-0.4240	-0.0118	-0.0437	0.0075	-5.2565

### 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	7.1010
Shear X	-0.0233
Shear Z	0.1173
Moment X	0.4439
Moment Y (Twist)	0.0168
Moment Z	9.1556

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.9871
Shear X	-0.0131
Shear Z	0.0812
Moment X	0.3060
Moment Y (Twist)	0.0095
Moment Z	5.2565

## 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	-0.0000	2.3553	-0.0044	-0.0168	0.0002	0.0212
ULS: 2. D + L	-0.0000	2.3553	-0.0044	-0.0168	0.0002	0.0212
ULS: 3. D + (S or Lr or R)	-0.0001	4.4439	-0.0091	-0.0348	0.0003	0.0214
ULS: 3. D + (S or Lr or R)	-0.0000	2.3553	-0.0044	-0.0168	0.0002	0.0212
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.9217	-0.0080	-0.0303	0.0003	0.0214
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.3553	-0.0044	-0.0168	0.0002	0.0212
ULS: 5b. D + 0.7E	-0.0000	2.3553	-0.0044	-0.0168	0.0002	0.0212

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.9217	-0.0080	-0.0303	0.0003	0.0214
ULS: 8. 0.6D + 0.7E	-0.0000	1.4132	-0.0027	-0.0101	0.0001	0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0131	4.7852	-0.0099	-0.0372	0.0073	-5.4930
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0131	4.7852	-0.0099	-0.0372	0.0073	-5.4930
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0130	4.7846	-0.0097	-0.0376	-0.0066	5.5359
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0128	0.4118	-0.0001	0.0001	0.0068	-5.5816
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0099	5.7441	-0.0121	-0.0456	0.0057	-4.1143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0099	5.7441	-0.0121	-0.0456	0.0057	-4.1143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0097	5.7437	-0.0119	-0.0459	-0.0048	4.1574
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0096	2.4641	-0.0047	-0.0175	0.0053	-4.1808
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0098	4.1777	-0.0086	-0.0321	0.0055	-4.1145
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0098	4.1777	-0.0086	-0.0321	0.0055	-4.1145
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0097	4.1773	-0.0084	-0.0324	-0.0049	4.1572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0096	0.8976	-0.0012	-0.0041	0.0051	-4.1809
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0131	3.8430	-0.0082	-0.0305	0.0072	-5.5015
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0131	3.8430	-0.0082	-0.0305	0.0072	-5.5015
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0130	3.8425	-0.0080	-0.0309	-0.0066	5.5274
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0128	-0.5303	0.0017	0.0069	0.0067	-5.5901

#### 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	8.1931
Shear X	-0.0233
Shear Z	-0.0175
Moment X	-0.0670
Moment Y (Twist)	0.0130
Moment Z	9.7623

#### 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	5.7441
Shear X	-0.0131
Shear Z	-0.0121
Moment X	-0.0459
Moment Y (Twist)	0.0073
Moment Z	5.5901

#### 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	-0.0000	2.3553	0.0044	0.0168	-0.0002	0.0212
ULS: 2. D + L	-0.0000	2.3553	0.0044	0.0168	-0.0002	0.0212
ULS: 3. D + (S or Lr or R)	-0.0001	4.4439	0.0091	0.0348	-0.0003	0.0214
ULS: 3. D + (S or Lr or R)	-0.0000	2.3553	0.0044	0.0168	-0.0002	0.0212
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.9217	0.0080	0.0303	-0.0003	0.0214
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.3553	0.0044	0.0168	-0.0002	0.0212
ULS: 5b. D + 0.7E	-0.0000	2.3553	0.0044	0.0168	-0.0002	0.0212
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.9217	0.0080	0.0303	-0.0003	0.0214
ULS: 8. 0.6D + 0.7E	-0.0000	1.4132	0.0027	0.0101	-0.0001	0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.0131	4.7852	0.0099	0.0372	-0.0073	-5.4930
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0131	4.7852	0.0099	0.0372	-0.0073	-5.4930
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0130	4.7846	0.0097	0.0376	0.0066	5.5359
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0128	0.4118	0.0001	-0.0001	-0.0068	-5.5816
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.0099	5.7441	0.0121	0.0456	-0.0057	-4.1143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0099	5.7441	0.0121	0.0456	-0.0057	-4.1143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0097	5.7437	0.0119	0.0459	0.0048	4.1574
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0096	2.4641	0.0047	0.0175	-0.0053	-4.1808

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	-0.0098	4.1777	0.0086	0.0321	-0.0055	-4.1145
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0098	4.1777	0.0086	0.0321	-0.0055	-4.1145
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0097	4.1773	0.0084	0.0324	0.0049	4.1572
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0096	0.8976	0.0012	0.0041	-0.0051	-4.1809
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.0131	3.8430	0.0082	0.0305	-0.0072	-5.5015
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0131	3.8430	0.0082	0.0305	-0.0072	-5.5015
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0130	3.8425	0.0080	0.0309	0.0066	5.5274
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0128	-0.5303	-0.0017	-0.0069	-0.0067	-5.5901

#### 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	8.1931
Shear X	-0.0233
Shear Z	0.0175
Moment X	0.0670
Moment Y (Twist)	0.0130
Moment Z	9.7623

#### 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	5.7441
Shear X	-0.0131
Shear Z	0.0121
Moment X	0.0459
Moment Y (Twist)	0.0073
Moment Z	5.5901

#### 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	0.0000	2.0762	-0.0297	-0.1120	0.0010	0.0206
ULS: 2. D + L	0.0000	2.0762	-0.0297	-0.1120	0.0010	0.0206
ULS: 3. D + (S or Lr or R)	0.0001	3.8702	-0.0612	-0.2308	0.0021	0.0204
ULS: 3. D + (S or Lr or R)	0.0000	2.0762	-0.0297	-0.1120	0.0010	0.0206
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.4217	-0.0533	-0.2011	0.0018	0.0205
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0762	-0.0297	-0.1120	0.0010	0.0206
ULS: 5b. D + 0.7E	0.0000	2.0762	-0.0297	-0.1120	0.0010	0.0206
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.4217	-0.0533	-0.2011	0.0018	0.0205
ULS: 8. 0.6D + 0.7E	0.0000	1.2457	-0.0178	-0.0672	0.0006	0.0124
ULS: 5a. D + 0.6W_Wind downforce Case A only	0.0131	4.1629	-0.0659	-0.2496	-0.0050	-5.1352
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0131	4.1629	-0.0659	-0.2496	-0.0050	-5.1352
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0130	4.1635	-0.0669	-0.2519	0.0095	5.1760
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0128	0.4065	-0.0001	-0.0011	-0.0071	-5.2482
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0099	4.9867	-0.0805	-0.3042	-0.0027	-3.8464
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0099	4.9867	-0.0805	-0.3042	-0.0027	-3.8464
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0097	4.9871	-0.0812	-0.3060	0.0082	3.8870
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0096	2.1694	-0.0311	-0.1179	-0.0043	-3.9312
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0098	3.6412	-0.0569	-0.2152	-0.0035	-3.8463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0098	3.6412	-0.0569	-0.2152	-0.0035	-3.8463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0097	3.6417	-0.0576	-0.2170	0.0074	3.8871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0096	0.8239	-0.0075	-0.0289	-0.0051	-3.9310
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	0.0131	3.3324	-0.0541	-0.2048	-0.0054	-5.1435
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0131	3.3324	-0.0541	-0.2048	-0.0054	-5.1435
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0130	3.3330	-0.0550	-0.2071	0.0091	5.1677
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0128	-0.4240	0.0118	0.0437	-0.0075	-5.2565

#### Worst Case Reactions LRFD

#### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.1010
Shear X	-0.0233
Shear Z	-0.1173
Moment X	-0.4440
Moment Y (Twist)	0.0168
Moment Z	9.1556

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

Result	Value (kip, kip-ft)
Axial	4.9871
Shear X	-0.0131
Shear Z	-0.0812
Moment X	-0.3060
Moment Y (Twist)	0.0095
Moment Z	5.2565

## Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States

User Name: sales@mtsolar.us  
 Unit System: imperial

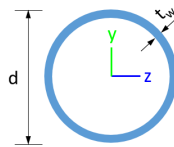


## Design Input Information

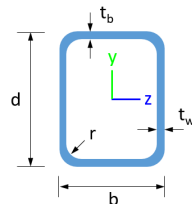
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

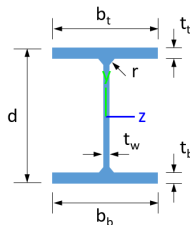
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



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



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

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28







103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.19	6.45	30.09	45.74
114	120.60	98.23	18.18	6.45	30.09	45.74
115	120.60	68.63	15.20	6.45	30.09	45.74
116	120.60	68.63	15.20	6.45	30.09	45.74
201	251.16	69.52	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	117.88	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	117.88	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	18.19	6.45	30.09	45.74
214	120.60	98.23	18.18	6.45	30.09	45.74
215	120.60	68.63	15.55	6.45	30.09	45.74
216	120.60	68.63	15.56	6.45	30.09	45.74
301	251.16	69.52	42.30	42.30	75.35	75.35
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	4.60	29.14	16.61
304	79.65	72.01	10.99	4.60	29.14	16.61
305	79.65	73.44	10.99	4.60	29.14	16.61
306	79.65	74.02	10.99	4.60	29.14	16.61
307	79.65	73.44	10.99	4.60	29.14	16.61
308	120.60	54.44	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	4.60	29.14	16.61
311	120.60	54.44	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	98.23	19.35	6.45	30.09	45.74
314	120.60	98.23	19.25	6.45	30.09	45.74
315	120.60	68.63	15.14	6.45	30.09	45.74
316	120.60	68.63	15.13	6.45	30.09	45.74

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	φ	Status
1	0.102	0.216	0.023	0.000	0.002	0.273	#15	0.673	Not Required	Pass
2	0.001	0.351	0.002	0.075	0.000	0.352	#21	0.034	Not Required	Pass
3	0.000	0.696	0.016	0.070	0.003	0.703	#15	0.044	Not Required	Pass
4	0.000	0.604	0.014	0.070	0.003	0.706	#15	0.117	Not Required	Pass

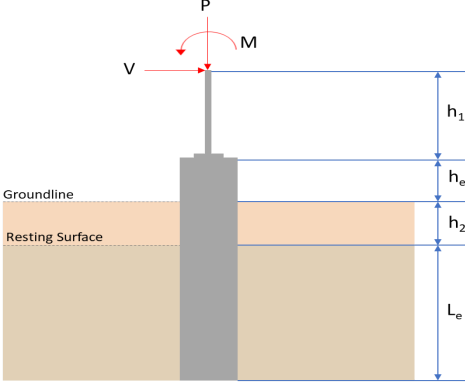
4	0.000	0.094	0.014	0.070	0.002	0.700	#13	0.117	Not Required	Pass
5	0.000	0.430	0.009	0.070	0.002	0.438	#15	0.073	Not Required	Pass
6	0.001	0.773	0.014	0.079	0.002	0.782	#15	0.044	Not Required	Pass
7	0.000	0.480	0.023	0.078	0.004	0.483	#15	0.073	Not Required	Pass
8	0.001	0.088	0.012	0.049	0.000	0.095	#13	0.088	Not Required	Pass
9	0.001	0.077	0.008	0.002	0.001	0.081	#23	0.198	Not Required	Pass
10	0.000	0.772	0.024	0.078	0.003	0.778	#13	0.117	Not Required	Pass
11	0.001	0.087	0.012	0.049	0.000	0.094	#15	0.088	Not Required	Pass
12	0.001	0.410	0.004	0.084	0.001	0.411	#23	0.052	Not Required	Pass
13	0.001	0.207	0.015	0.064	0.001	0.213	#15	0.265	Not Required	Pass
14	0.001	0.211	0.015	0.064	0.001	0.216	#13	0.177	Not Required	Pass
15	0.000	0.075	0.000	0.031	0.000	0.075	#15	Not Required	Not Required	Pass
16	0.000	0.075	0.000	0.031	0.000	0.075	#13	Not Required	Not Required	Pass
101	0.118	0.236	0.003	0.000	0.000	0.288	#15	0.673	Not Required	Pass
102	0.000	0.447	0.003	0.093	0.001	0.447	#21	0.052	Not Required	Pass
103	0.000	0.849	0.005	0.086	0.001	0.854	#15	0.044	Not Required	Pass
104	0.000	0.848	0.015	0.086	0.002	0.858	#13	0.117	Not Required	Pass
105	0.000	0.526	0.016	0.085	0.002	0.530	#15	0.073	Not Required	Pass
106	0.000	0.845	0.006	0.086	0.002	0.851	#15	0.044	Not Required	Pass
107	0.000	0.524	0.003	0.085	0.001	0.525	#15	0.073	Not Required	Pass
108	0.001	0.065	0.000	0.050	0.000	0.065	#13	0.088	Not Required	Pass
109	0.001	0.078	0.003	0.001	0.000	0.080	#15	0.198	Not Required	Pass
110	0.000	0.844	0.003	0.085	0.000	0.848	#13	0.117	Not Required	Pass
111	0.001	0.064	0.000	0.050	0.000	0.064	#15	0.088	Not Required	Pass
112	0.000	0.438	0.002	0.092	0.000	0.438	#21	0.052	Not Required	Pass
113	0.001	0.232	0.015	0.066	0.000	0.233	#15	0.265	Not Required	Pass
114	0.001	0.236	0.014	0.066	0.000	0.236	#13	0.265	Not Required	Pass
115	0.001	0.249	0.011	0.052	0.000	0.251	#15	0.439	Not Required	Pass
116	0.001	0.251	0.011	0.052	0.000	0.252	#13	0.439	Not Required	Pass
201	0.118	0.236	0.003	0.000	0.000	0.288	#15	0.673	Not Required	Pass
202	0.000	0.438	0.002	0.092	0.000	0.438	#21	0.052	Not Required	Pass
203	0.000	0.845	0.006	0.086	0.002	0.851	#15	0.044	Not Required	Pass
204	0.000	0.844	0.003	0.085	0.000	0.848	#13	0.117	Not Required	Pass
205	0.000	0.524	0.003	0.085	0.001	0.525	#15	0.073	Not Required	Pass
206	0.000	0.849	0.005	0.086	0.001	0.854	#15	0.044	Not Required	Pass
207	0.000	0.526	0.016	0.085	0.002	0.530	#15	0.073	Not Required	Pass
208	0.001	0.075	0.011	0.052	0.000	0.082	#13	0.088	Not Required	Pass
209	0.001	0.078	0.003	0.001	0.000	0.080	#15	0.198	Not Required	Pass
210	0.000	0.848	0.015	0.086	0.002	0.858	#13	0.117	Not Required	Pass
211	0.001	0.074	0.011	0.052	0.000	0.082	#15	0.088	Not Required	Pass
212	0.000	0.447	0.003	0.093	0.001	0.447	#21	0.052	Not Required	Pass
213	0.001	0.232	0.015	0.066	0.000	0.233	#15	0.265	Not Required	Pass
214	0.001	0.236	0.014	0.066	0.000	0.236	#13	0.265	Not Required	Pass
215	0.002	0.224	0.000	0.050	0.000	0.225	#15	0.439	Not Required	Pass
216	0.002	0.225	0.000	0.050	0.000	0.226	#13	0.439	Not Required	Pass
301	0.102	0.216	0.023	0.000	0.002	0.273	#15	0.673	Not Required	Pass
302	0.001	0.410	0.004	0.084	0.001	0.411	#23	0.052	Not Required	Pass
303	0.001	0.773	0.014	0.079	0.002	0.782	#15	0.044	Not Required	Pass
304	0.000	0.772	0.024	0.078	0.003	0.778	#13	0.117	Not Required	Pass
305	0.000	0.480	0.023	0.078	0.004	0.483	#15	0.073	Not Required	Pass
306	0.000	0.696	0.016	0.070	0.003	0.703	#15	0.044	Not Required	Pass
307	0.000	0.430	0.009	0.070	0.002	0.438	#15	0.073	Not Required	Pass
308	0.000	0.075	0.000	0.031	0.000	0.075	#13	Not Required	Not Required	Pass
309	0.001	0.077	0.008	0.002	0.001	0.081	#23	0.198	Not Required	Pass

310	0.000	0.694	0.014	0.070	0.002	0.706	#13	0.117	Not Required	Pass
311	0.000	0.075	0.000	0.031	0.000	0.075	#15	Not Required	Not Required	Pass
312	0.001	0.351	0.002	0.075	0.000	0.352	#21	0.034	Not Required	Pass
313	0.001	0.207	0.015	0.064	0.001	0.213	#15	0.177	Not Required	Pass
314	0.001	0.211	0.015	0.064	0.001	0.216	#13	0.265	Not Required	Pass
315	0.001	0.256	0.012	0.049	0.000	0.255	#15	0.439	Not Required	Pass
316	0.001	0.257	0.012	0.049	0.000	0.257	#13	0.439	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	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
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: round <math>D = 36</math> in - Pile diameter <math>L = 6.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resisting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.987</td> <td>7.101</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.013</td> <td>-0.023</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.081</td> <td>0.117</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.306</td> <td>0.444</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>5.256</td> <td>9.156</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.987	7.101	$V_x$ (kip)	-0.013	-0.023	$V_z$ (kip)	0.081	0.117	$M_x$ (kipft)	0.306	0.444	$M_z$ (kipft)	5.256	9.156	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
Load Component	ASD	LRFD																										
$P$ (kip)	4.987	7.101																										
$V_x$ (kip)	-0.013	-0.023																										
$V_z$ (kip)	0.081	0.117																										
$M_x$ (kipft)	0.306	0.444																										
$M_z$ (kipft)	5.256	9.156																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.013 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.0043333 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(5.256 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.752 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$$L_{e,x} = 6.0158 \text{ ft} - \text{Required depth in x-direction,}$$

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.027 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.306 \text{ kipft}) + ((0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.102 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$$L_{e,z} = 2.7004 \text{ ft} - \text{Required depth in z-direction,}$$

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.0158 \text{ ft}), (2.7004 \text{ ft})]$$

$$L_{e,req} = 6.016 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.016 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.96256$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(4.987 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.70552 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.70552 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35276$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

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

$M_o = 1.752 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (1.752 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.172 \text{ ft}$$

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

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

$$p = \frac{1.178 [(4 \times (1.752 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (1.752 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.27816 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 [(2 \times (1.752 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.83891 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.172 \text{ ft})}{2}$$

$$p_a = 0.3129 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27816 \text{ kip/ft}^2)}{(0.3129 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88896$$

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

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.83891 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89484$$

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

**Considering z-direction:**

$H_o = 0.027 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.102 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.102 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.027 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.102 \text{ kipft/ft})) + (4 \times (0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4398 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.102 \text{ kipft/ft})) + (3 \times (0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.102 \text{ kipft/ft})) + (2 \times (0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.039171 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (0.102 \text{ kipft/ft})) + ((0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.089937 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4398 \text{ ft})}{2}$$

$$p_a = 0.33299 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.039171 \text{ kip/ft}^2)}{(0.33299 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.11764$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

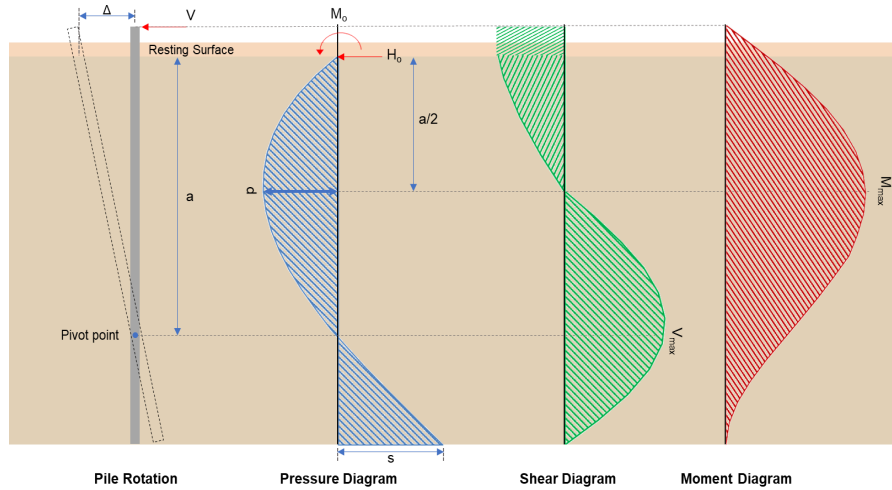
Status: **PASS**  
Ratio: **0.120**

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

$$Ratio = \frac{(0.089937 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.095933$$

Status: **PASS**  
Ratio: **0.100**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0076667 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.156 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.052 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.052 \text{ kipft/ft})}{(-0.0076667 \text{ kip/ft})}$$

$$E = 398.09 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.052 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.052 \text{ kipft/ft})) + (4 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1721 \text{ ft}$$

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

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

$$V_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1721 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1721 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.6189 \text{ kip}$$

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

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

$$M_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(398.09 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.1707 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.117 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.039 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.444 \text{ kipft}) + ((0.117 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.148 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.148 \text{ kipft/ft})}{(0.039 \text{ kip/ft})}$$

$$E = 3.7949 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.148 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.039 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.148 \text{ kipft/ft})) + (4 \times (0.039 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4392 \text{ ft}$$

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

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

$$V_{max} = ((0.039 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4392 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4392 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20344 \text{ kip}$$

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

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

$$M_{max} = ((0.039 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.7949 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.57034 \text{ kipft}$$

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

Table 22.4.2.1

**Longitudinal reinforcement:**

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.101 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.151 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.151 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

**Ratio** - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

**Ties:**

25.7.2.2

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

25.7.2.1

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

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

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

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

**Summary:**

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

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

**Axial Compression Strength (ACI 318-19, LFRD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.101 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0056631$$

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

**Shear Strength (ACI 318-19, LFRD)**

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.101 \text{ kip} \rightarrow 7101 \text{ lbf}$ .  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7101 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.643 \text{ kip}$$

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)

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

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

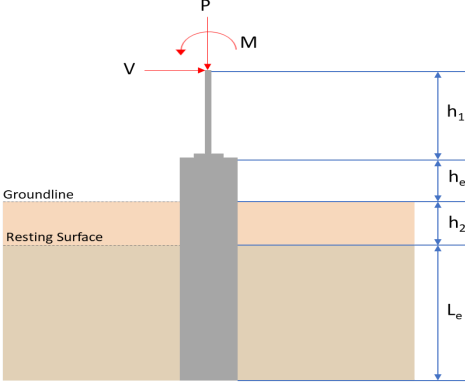
$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.643 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 75.643 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.643 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.979 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 2.6189 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(2.6189 \text{ kip})}{(73.979 \text{ kip})}$ $Ratio = 0.0354$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.20344 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.20344 \text{ kip})}{(73.979 \text{ kip})}$ $Ratio = 0.0027499$	<p>Status: <b>PASS</b> Ratio: <b>0.040</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></p> <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),          Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 8.1707 \text{ kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(8.1707 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.13173$	<p>Status: <b>PASS</b>          Ratio: <b>0.130</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.57034 \text{ kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.57034 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.009195$	<p>Status: <b>PASS</b>          Ratio: <b>0.010</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: round <math>D = 36</math> in - Pile diameter <math>L = 6.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resisting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.744</td> <td>8.193</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.013</td> <td>-0.023</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.012</td> <td>-0.018</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.046</td> <td>-0.067</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>5.590</td> <td>9.762</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> 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)	5.744	8.193	$V_x$ (kip)	-0.013	-0.023	$V_z$ (kip)	-0.012	-0.018	$M_x$ (kipft)	-0.046	-0.067	$M_z$ (kipft)	5.590	9.762	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.013 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.0043333 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(5.59 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.8633 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.1416 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.012 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.004 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.046 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.015333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.1435 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1416 \text{ ft}), (1.1435 \text{ ft})]$$

$$L_{e,req} = 6.142 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.142 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.98272$$

Status: **PASS**  
Ratio: **0.980**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.744 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.81261 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.81261 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.4063$$

Status: **PASS**  
Ratio: **0.410**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

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

$M_o = 1.8633 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8633 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (1.8633 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1717 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.8633 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (1.8633 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.29606 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (1.8633 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89264 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1717 \text{ ft})}{2}$$

$$p_a = 0.31287 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29606 \text{ kip/ft}^2)}{(0.31287 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94626$$

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

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89264 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95214$$

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

**Considering z-direction:**

$H_o = -0.004 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.015333 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015333 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.015333 \text{ kipft/ft})) + (4 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4379 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.015333 \text{ kipft/ft})) + (3 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.015333 \text{ kipft/ft})) + (2 \times (-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0014082 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (0.015333 \text{ kipft/ft})) + ((-0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0013673 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4379 \text{ ft})}{2}$$

$$p_a = 0.33285 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0014082 \text{ kip/ft}^2)}{(0.33285 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0042307$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

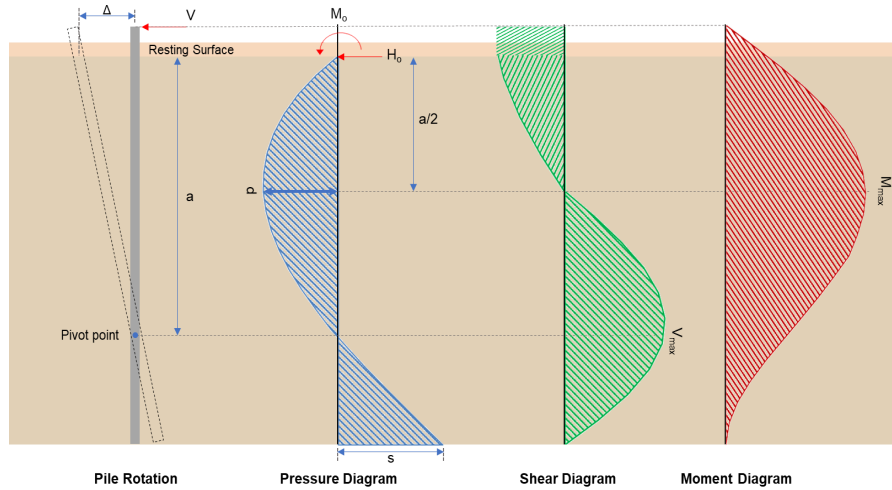
Status: **PASS**  
Ratio: **0.000**

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

$$Ratio = \frac{(0.0013673 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0014584$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0076667 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.762 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.254 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.254 \text{ kipft/ft})}{(-0.0076667 \text{ kip/ft})}$$

$$E = 424.43 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.254 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.254 \text{ kipft/ft})) + (4 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1717 \text{ ft}$$

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

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

$$V_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1717 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1717 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.7912 \text{ kip}$$

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

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

$$M_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(424.43 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.7093 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

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

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

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.022333 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.022333 \text{ kipft/ft})}{(-0.006 \text{ kip/ft})}$$

$$E = 3.7222 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.022333 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.006 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.022333 \text{ kipft/ft})) + (4 \times (-0.006 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4418 \text{ ft}$$

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

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

$$V_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4418 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4418 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.030931 \text{ kip}$$

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

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

$$M_{max} = ((-0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.7222 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.0866 \text{ kipft}$$

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.193 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.117 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.117 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

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

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

**Axial Compression Strength (ACI 318-19, LFRD)**

22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.193 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.006534$$

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

**Shear Strength (ACI 318-19, LFRD)**

**Parameters:**

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

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.193 \text{ kip} \rightarrow 8193 \text{ lbf}$ .  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8193 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.829 \text{ kip}$$

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)

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

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

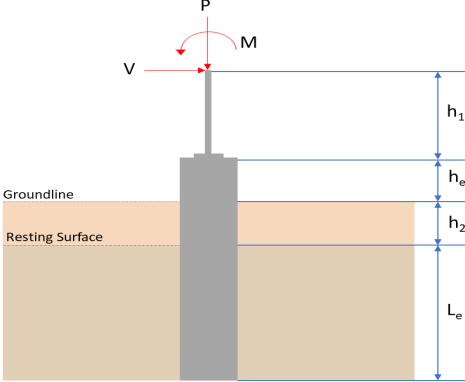
$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.829 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 75.829 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.829 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.099 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 2.7912 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(2.7912 \text{ kip})}{(74.099 \text{ kip})}$ $Ratio = 0.037669$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.030931 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.030931 \text{ kip})}{(74.099 \text{ kip})}$ $Ratio = 0.00041742$	<p>Status: <b>PASS</b> Ratio: <b>0.040</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></p> <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 8.7093 \text{ kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(8.7093 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.14041$	<p>Status: <b>PASS</b>  Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.0866 \text{ kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.0866 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0013962$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: round <math>D = 36</math> in - Pile diameter <math>L = 6.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resisting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.744</td> <td>8.193</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.013</td> <td>-0.023</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.012</td> <td>0.018</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.046</td> <td>0.067</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>5.590</td> <td>9.762</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> 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)	5.744	8.193	$V_x$ (kip)	-0.013	-0.023	$V_z$ (kip)	0.012	0.018	$M_x$ (kipft)	0.046	0.067	$M_z$ (kipft)	5.590	9.762	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.013 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.0043333 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(5.59 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.8633 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.1416 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.012 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.004 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.046 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.015333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.3451 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1416 \text{ ft}), (1.3451 \text{ ft})]$$

$$L_{e,req} = 6.142 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.142 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.98272$$

Status: **PASS**  
Ratio: **0.980**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.744 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.81261 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.81261 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.4063$$

Status: **PASS**  
Ratio: **0.410**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

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

$M_o = 1.8633 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8633 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (1.8633 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1717 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (1.8633 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (1.8633 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.29606 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (1.8633 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89264 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1717 \text{ ft})}{2}$$

$$p_a = 0.31287 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29606 \text{ kip/ft}^2)}{(0.31287 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94626$$

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

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89264 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95214$$

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

#### Considering z-direction:

$H_o = 0.004 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.015333 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015333 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.015333 \text{ kipft/ft})) + (4 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4379 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.015333 \text{ kipft/ft})) + (3 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.015333 \text{ kipft/ft})) + (2 \times (0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0058387 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (0.015333 \text{ kipft/ft})) + ((0.004 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.013431 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4379 \text{ ft})}{2}$$

$$p_a = 0.33285 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0058387 \text{ kip/ft}^2)}{(0.33285 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.017542$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

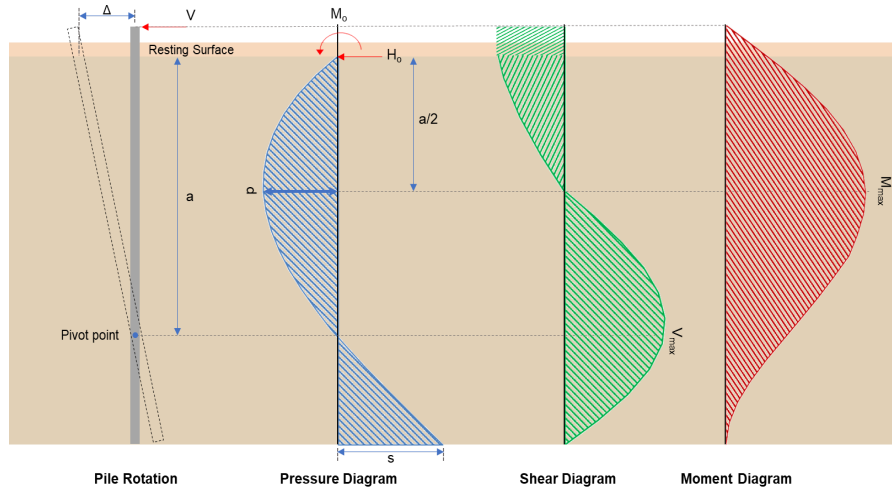
Status: **PASS**  
Ratio: **0.020**

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

$$Ratio = \frac{(0.013431 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.014327$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0076667 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.762 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.254 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.254 \text{ kipft/ft})}{(-0.0076667 \text{ kip/ft})}$$

$$E = 424.43 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.254 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.254 \text{ kipft/ft})) + (4 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1717 \text{ ft}$$

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

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

$$V_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1717 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1717 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.7912 \text{ kip}$$

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

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

$$M_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(424.43 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (424.43 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1717 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.7093 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

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

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

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.022333 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.022333 \text{ kipft/ft})}{(0.006 \text{ kip/ft})}$$

$$E = 3.7222 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.022333 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.006 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.022333 \text{ kipft/ft})) + (4 \times (0.006 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4418 \text{ ft}$$

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

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

$$V_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4418 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4418 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.030931 \text{ kip}$$

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

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

$$M_{max} = ((0.006 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.7222 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.7222 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.0866 \text{ kipft}$$

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.193 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.117 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.117 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

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

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

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.193 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.006534$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.193 \text{ kip} \rightarrow 8193 \text{ lbf}$ .  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(8193 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.829 \text{ kip}$$

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)

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

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

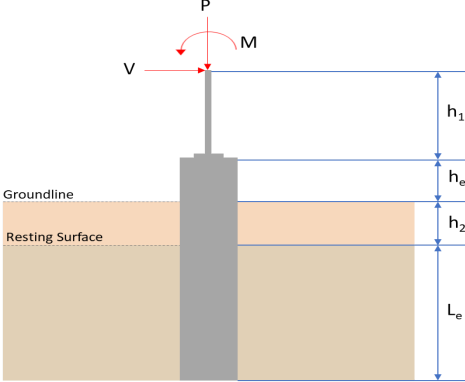
$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.829 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 75.829 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.829 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.099 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 2.7912 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(2.7912 \text{ kip})}{(74.099 \text{ kip})}$ $Ratio = 0.037669$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.030931 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.030931 \text{ kip})}{(74.099 \text{ kip})}$ $Ratio = 0.00041742$	<p>Status: <b>PASS</b> Ratio: <b>0.040</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></p> <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),          Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 8.7093 \text{ kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(8.7093 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.14041$	<p>Status: <b>PASS</b>          Ratio: <b>0.140</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.0866 \text{ kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.0866 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0013962$	<p>Status: <b>PASS</b>          Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: round <math>D = 36</math> in - Pile diameter <math>L = 6.25</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resisting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.987</td> <td>7.101</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.013</td> <td>-0.023</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.081</td> <td>-0.117</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.306</td> <td>-0.444</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>5.256</td> <td>9.156</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.987	7.101	$V_x$ (kip)	-0.013	-0.023	$V_z$ (kip)	-0.081	-0.117	$M_x$ (kipft)	-0.306	-0.444	$M_z$ (kipft)	5.256	9.156	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
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Load Component	ASD	LRFD																										
$P$ (kip)	4.987	7.101																										
$V_x$ (kip)	-0.013	-0.023																										
$V_z$ (kip)	-0.081	-0.117																										
$M_x$ (kipft)	-0.306	-0.444																										
$M_z$ (kipft)	5.256	9.156																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.013 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.0043333 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(5.256 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 1.752 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.0158 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.027 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.306 \text{ kipft}) + ((-0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.102 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.9811 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.0158 \text{ ft}), (1.9811 \text{ ft})]$$

$$L_{e,req} = 6.016 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.016 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.96256$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(4.987 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.70552 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.70552 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35276$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

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

$M_o = 1.752 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (1.752 \text{ kipft/ft})) + (4 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.172 \text{ ft}$$

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

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

$$p = \frac{1.178 [(4 \times (1.752 \text{ kipft/ft})) + (3 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (1.752 \text{ kipft/ft})) + (2 \times (-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.27816 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 [(2 \times (1.752 \text{ kipft/ft})) + ((-0.0043333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.83891 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.172 \text{ ft})}{2}$$

$$p_a = 0.3129 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27816 \text{ kip/ft}^2)}{(0.3129 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88896$$

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

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.83891 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89484$$

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

**Considering z-direction:**

$H_o = -0.027 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.102 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.102 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.027 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.102 \text{ kipft/ft})) + (4 \times (-0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4398 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.102 \text{ kipft/ft})) + (3 \times (-0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.102 \text{ kipft/ft})) + (2 \times (-0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0092414 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{9.425 \times [(2 \times (0.102 \text{ kipft/ft})) + ((-0.027 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0085051 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4398 \text{ ft})}{2}$$

$$p_a = 0.33299 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0092414 \text{ kip/ft}^2)}{(0.33299 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.027753$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

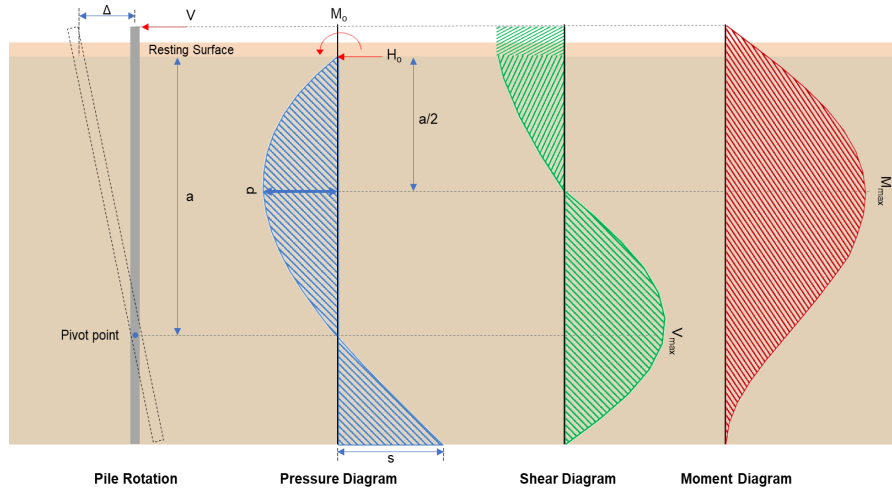
Status: **PASS**  
Ratio: **-0.030**

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

$$Ratio = \frac{(0.0085051 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0090721$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.0076667 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.156 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.052 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.052 \text{ kipft/ft})}{(-0.0076667 \text{ kip/ft})}$$

$$E = 398.09 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.052 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.052 \text{ kipft/ft})) + (4 \times (-0.0076667 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.1721 \text{ ft}$$

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

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

$$V_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1721 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1721 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 2.6189 \text{ kip}$$

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

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

$$M_{max} = ((-0.0076667 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(398.09 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (398.09 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.1721 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.1707 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.117 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.039 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.444 \text{ kipft}) + ((-0.117 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.148 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.148 \text{ kipft/ft})}{(-0.039 \text{ kip/ft})}$$

$$E = 3.7949 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.148 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.039 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.148 \text{ kipft/ft})) + (4 \times (-0.039 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4392 \text{ ft}$$

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

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

$$V_{max} = ((-0.039 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4392 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4392 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20344 \text{ kip}$$

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

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

$$M_{max} = ((-0.039 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.7949 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.7949 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4392 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.57034 \text{ kipft}$$

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

Table 22.4.2.1

**Longitudinal reinforcement:**

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.101 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.151 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.151 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

**Ratio** - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

**Ties:**

25.7.2.2

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

25.7.2.1

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

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

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

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

**Summary:**

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

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

**Axial Compression Strength (ACI 318-19, LFRD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[ (0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.101 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0056631$$

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

**Shear Strength (ACI 318-19, LFRD)**

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.101 \text{ kip} \rightarrow 7101 \text{ lbf}$ .  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7101 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.643 \text{ kip}$$

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)

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

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.643 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 75.643 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.643 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.979 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 2.6189 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(2.6189 \text{ kip})}{(73.979 \text{ kip})}$ $Ratio = 0.0354$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.20344 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.20344 \text{ kip})}{(73.979 \text{ kip})}$ $Ratio = 0.0027499$	<p>Status: <b>PASS</b> Ratio: <b>0.040</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></p> <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 8.1707 \text{ kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(8.1707 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.13173$	<p>Status: <b>PASS</b>  Ratio: <b>0.130</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.57034 \text{ kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.57034 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.009195$	<p>Status: <b>PASS</b>  Ratio: <b>0.010</b></p>