

# Your Project Calculations



Project Name: UnivofMNMorris-JB-RevA

S3D Model Link:  
[https://platform.skyciv.com/structural?preload\\_name=UnivofMNMorris-JB-RevA&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/9\\_2023](https://platform.skyciv.com/structural?preload_name=UnivofMNMorris-JB-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023)

Public Model Link:  
[https://platform.skyciv.com/structural-viewer?project\\_id=XfzlliriGppaByAH0LnC7v476nlbPjuVaEBCfFmLOTToc1YAz7KPIgcO4c2m1L](https://platform.skyciv.com/structural-viewer?project_id=XfzlliriGppaByAH0LnC7v476nlbPjuVaEBCfFmLOTToc1YAz7KPIgcO4c2m1L)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	5P-19.75-6TOP-HD-57-L-4Hx13W-LHAD
<b>Duty Classification:</b>	HD
<b>Module Width:</b>	44.65 in
<b>Module Length:</b>	89.72in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	13
<b>Total Number of Modules:</b>	52
<b>Desired Tilt Angle:</b>	30
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	15.48 ft
<b>Total Frame Length:</b>	96.00 ft
<b>Frame Weight:</b>	4675 lbs
<b>Array Dimensions N/S:</b>	15.05 ft
<b>Array Dimensions E/W:</b>	98.28 ft
<b>Rail Length:</b>	180.60 in
<b>Rail Spacing:</b>	3.78 ft
<b>Rail Check:</b>	PASS (56% utilized)

## Support Specifications

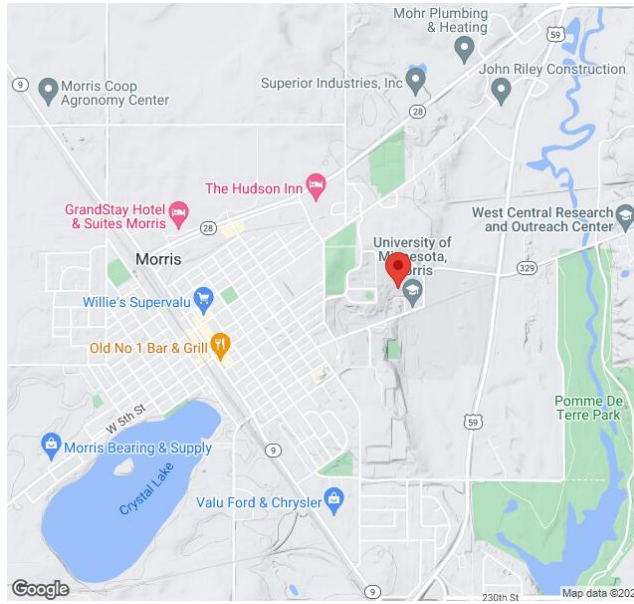
<b>Pole Size:</b>	6in Pipe Sch 80
<b>Pole Length above Grade:</b>	11.76 ft
<b>Number of Poles:</b>	5
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.25 ft Pile 2: 6.25 ft Pile 3: 6.50 ft Pile 4: 6.25 ft Pile 5: 6.25 ft
<b>Foundation Volume:</b>	18.667 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.143730 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	600 E 4th St, Morris, MN 56267, USA
<b>Wind Speed:</b>	104 mph
<b>Snow Load:</b>	50 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.021993 ksf



### Design Disclaimer

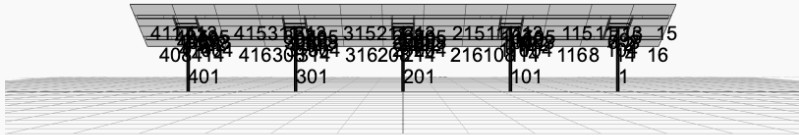
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

### AutoDesigner Input

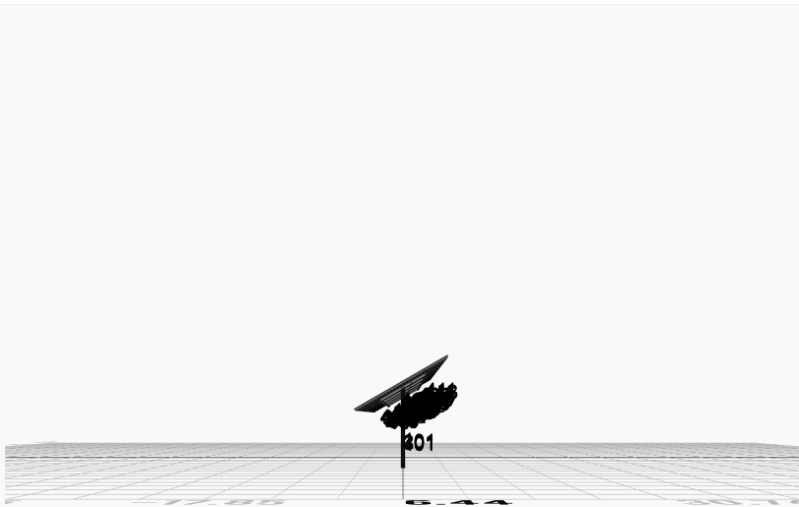
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### Design Notes:

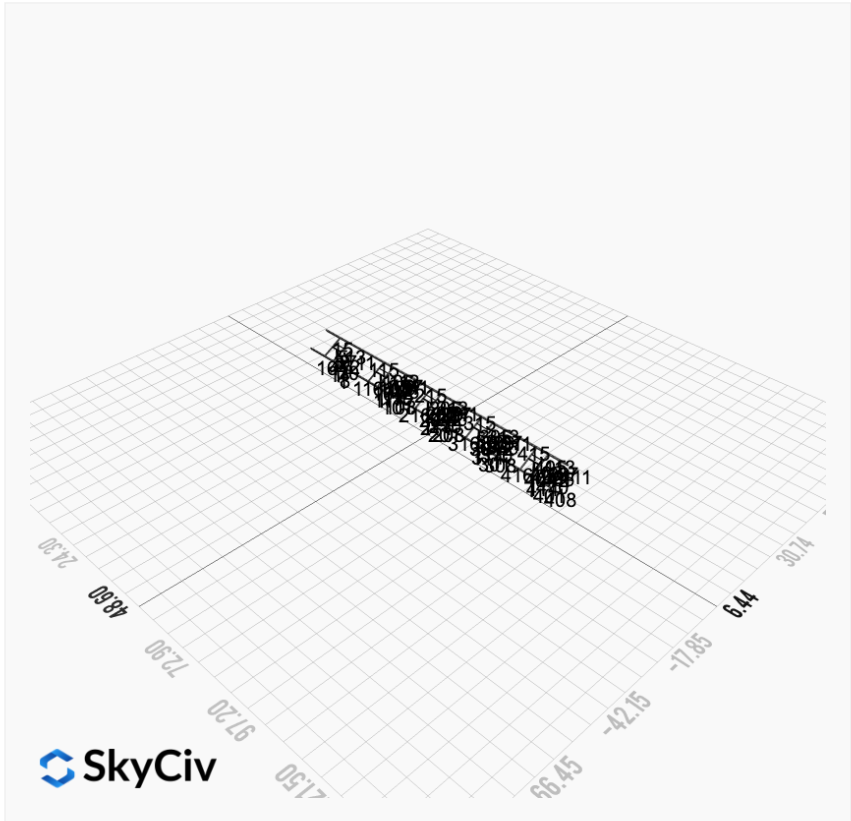
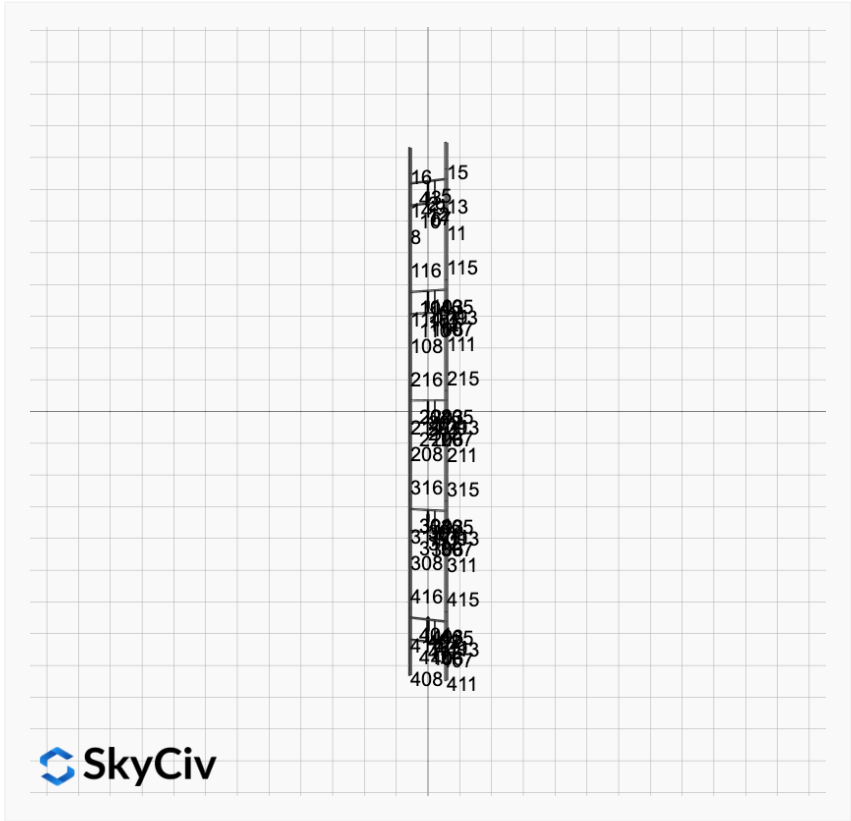
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles

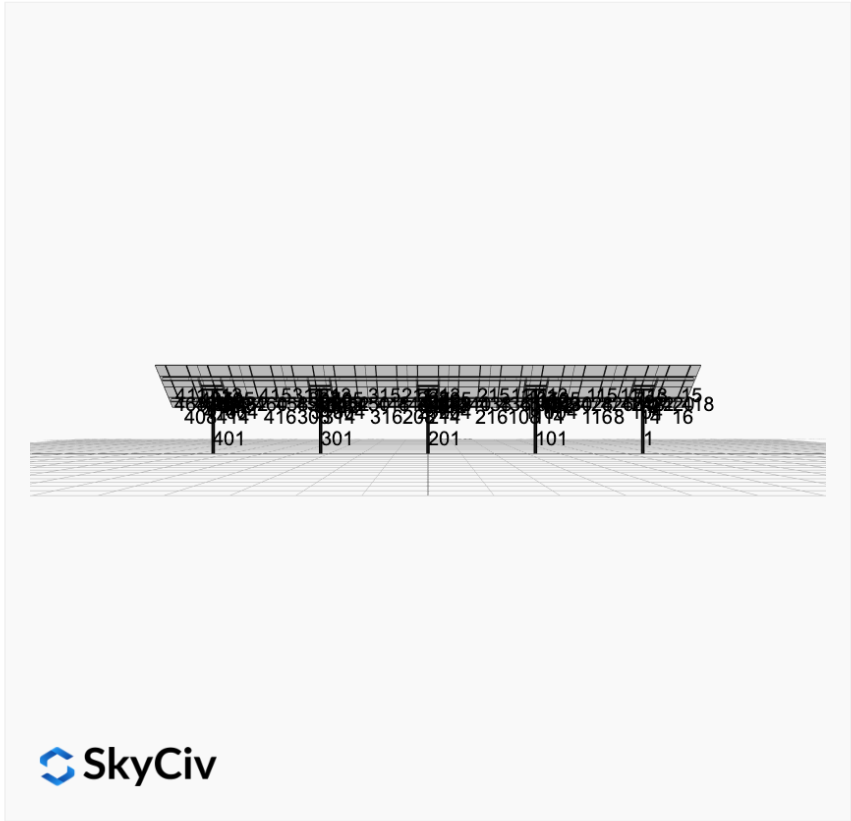


 SkyCiv



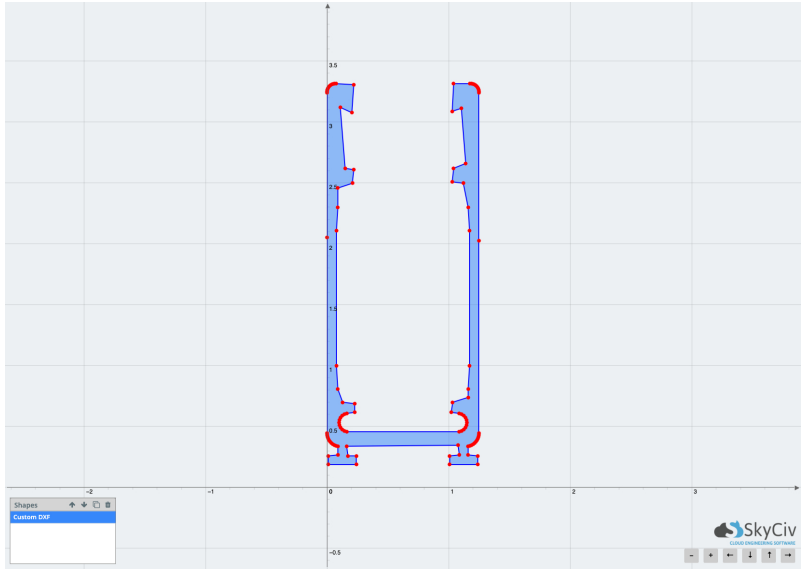
 SkyCiv





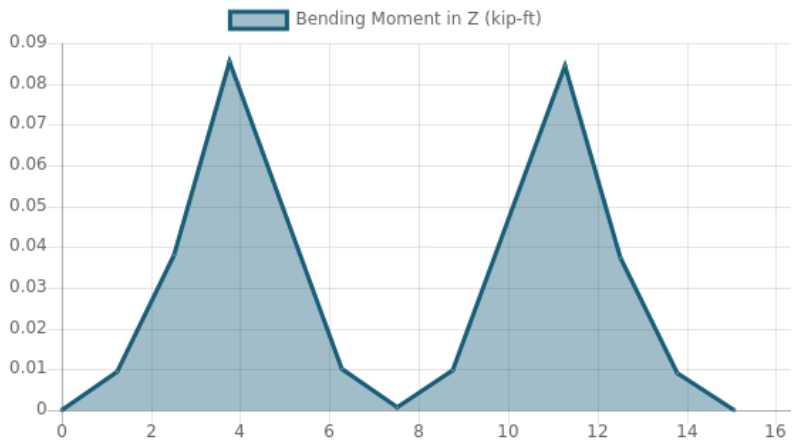
**Rail Design Check**

**Rail Length:** 15.049999999999999 ft  
**Additional Restraints Required:** None  
**Tributary Width:** 3.78 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Snow (X):** 0.0720 kip/ft  
**Snow (Y):** -0.0416 kip/ft  
**Wind uplift Case A:** 0.0751 kip/ft  
**Wind uplift Case A:** 0.0751 kip/ft  
**Wind uplift Case B (X):** 0.0000 kip/ft  
**Wind uplift Case B (Y):** 0.1043 kip/ft

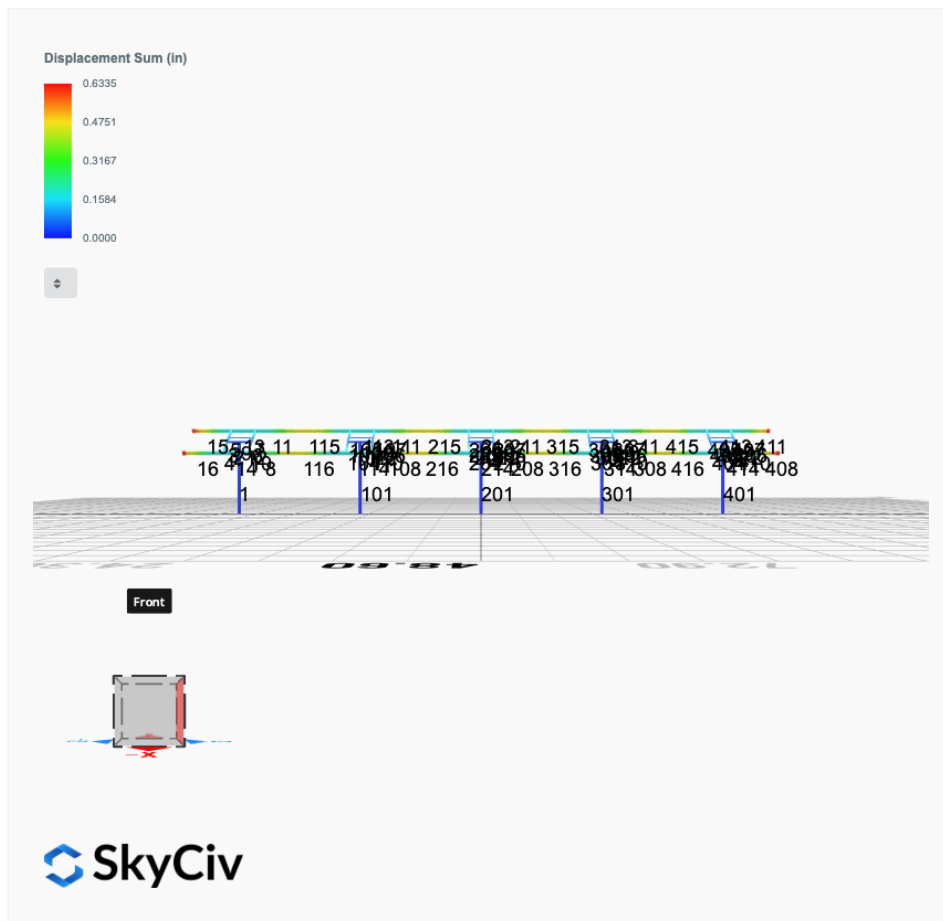


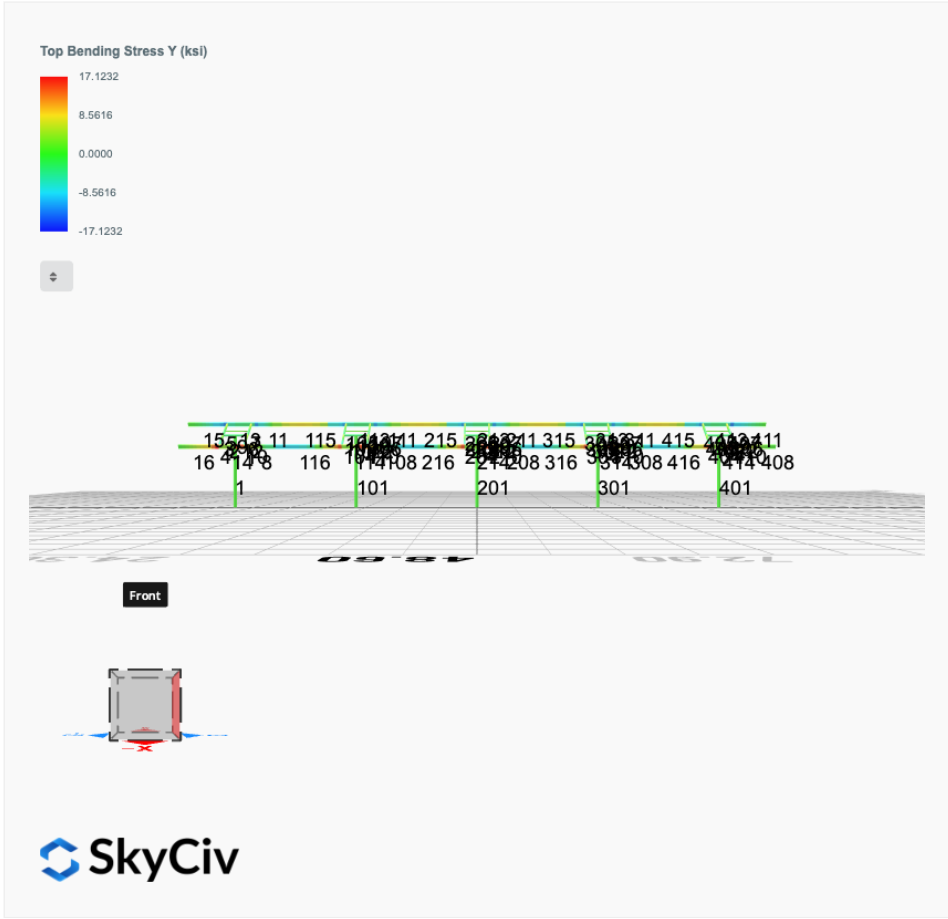
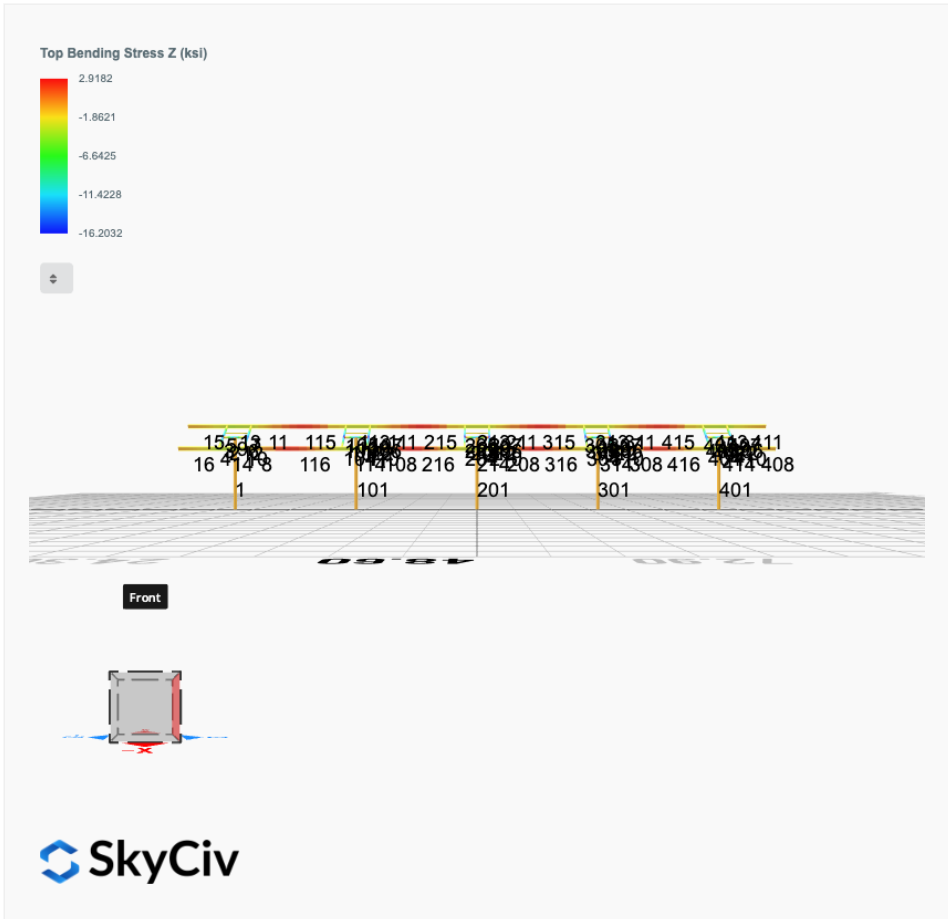
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	19.16725575	0.556	PASS
Material Yield	34.5	19.16725575	0.556	PASS
Material Strength	37	19.16725575	0.518	PASS

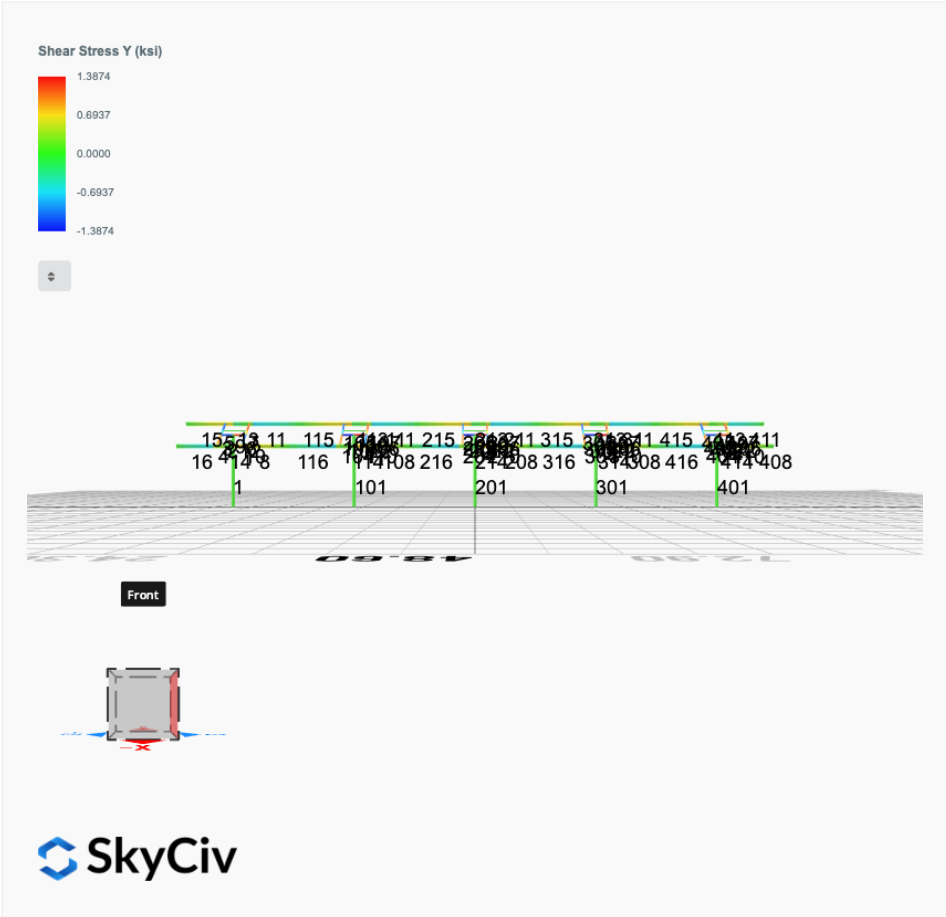
**Member 1, ULS: 1. 1.4D**

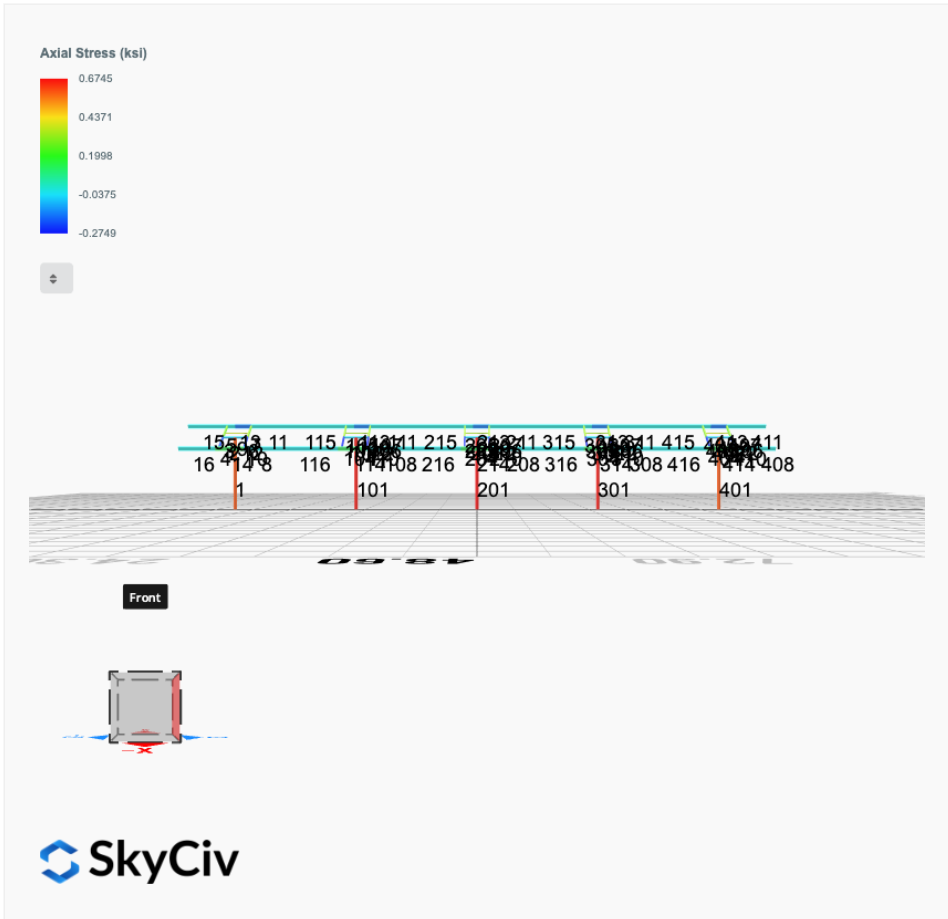


## FEM Results (Envelope Worst Case for each member)









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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0034	2.2072	-0.0041	-0.0135	0.0202	0.0627
ULS: 2. D + L	-0.0034	2.2072	-0.0041	-0.0135	0.0202	0.0627
ULS: 3. D + (S or Lr or R)	-0.0140	7.4967	-0.0166	-0.0552	0.0837	0.2100
ULS: 3. D + (S or Lr or R)	-0.0034	2.2072	-0.0041	-0.0135	0.0202	0.0627
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0114	6.1743	-0.0135	-0.0448	0.0678	0.1732
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0034	2.2072	-0.0041	-0.0135	0.0202	0.0627
ULS: 5b. D + 0.7E	-0.0034	2.2072	-0.0041	-0.0135	0.0202	0.0627
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0114	6.1743	-0.0135	-0.0448	0.0678	0.1732
ULS: 8. 0.6D + 0.7E	-0.0020	1.3243	-0.0024	-0.0081	0.0121	0.0376
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0009	5.6443	-0.0036	-0.0125	0.0110	24.3999
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0009	5.6443	-0.0036	-0.0125	0.0110	24.3999
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7072	-0.7383	-0.0031	-0.0096	0.0247	-19.5509
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4281	-0.2492	-0.0081	-0.0275	0.0360	-23.2970
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5095	8.7521	-0.0131	-0.0440	0.0610	18.4261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5095	8.7521	-0.0131	-0.0440	0.0610	18.4261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2716	3.9652	-0.0128	-0.0418	0.0712	-14.5370
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0623	4.3320	-0.0165	-0.0553	0.0797	-17.3466
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5015	4.7850	-0.0037	-0.0128	0.0133	18.3156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5015	4.7850	-0.0037	-0.0128	0.0133	18.3156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2795	-0.0019	-0.0033	-0.0106	0.0235	-14.6475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0702	0.3649	-0.0071	-0.0240	0.0320	-17.4571
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9996	4.7614	-0.0020	-0.0071	0.0030	24.3748
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9996	4.7614	-0.0020	-0.0071	0.0030	24.3748
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7085	-1.6212	-0.0015	-0.0042	0.0166	-19.5759
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4295	-1.1321	-0.0065	-0.0221	0.0279	-23.3221

### 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	13.9764
Shear X	-3.3404
Shear Z	-0.0303
Moment X	-0.1013
Moment Y (Twist)	0.1439
Moment Z	42.1221

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.7521
Shear X	-2.0009
Shear Z	-0.0166
Moment X	-0.0553
Moment Y (Twist)	0.0837
Moment Z	24.3999

## 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	0.0037	2.3154	0.0019	0.0070	-0.0039	-0.0093
ULS: 2. D + L	0.0037	2.3154	0.0019	0.0070	-0.0039	-0.0093
ULS: 3. D + (S or Lr or R)	0.0154	7.9505	0.0078	0.0292	-0.0161	-0.0892
ULS: 3. D + (S or Lr or R)	0.0037	2.3154	0.0019	0.0070	-0.0039	-0.0093
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0125	6.5417	0.0063	0.0236	-0.0131	-0.0692
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0037	2.3154	0.0019	0.0070	-0.0039	-0.0093
ULS: 5b. D + 0.7E	0.0037	2.3154	0.0019	0.0070	-0.0039	-0.0093

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0125	6.5417	0.0063	0.0236	-0.0131	-0.0692
ULS: 8. 0.6D + 0.7E	0.0022	1.3892	0.0011	0.0042	-0.0023	-0.0056
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0849	5.9480	0.0173	0.0624	-0.0417	25.4002
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0849	5.9480	0.0173	0.0624	-0.0417	25.4002
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7951	-0.7988	-0.0101	-0.0360	0.0254	-20.4576
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4923	-0.2778	-0.0120	-0.0427	0.0315	-24.2888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5540	9.2662	0.0179	0.0652	-0.0414	18.9879
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5540	9.2662	0.0179	0.0652	-0.0414	18.9879
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3560	4.2062	-0.0027	-0.0086	0.0089	-15.4054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1290	4.5969	-0.0041	-0.0136	0.0135	-18.2788
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5628	5.0398	0.0134	0.0485	-0.0322	19.0478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5628	5.0398	0.0134	0.0485	-0.0322	19.0478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3473	-0.0202	-0.0071	-0.0252	0.0181	-15.3455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1202	0.3705	-0.0086	-0.0303	0.0227	-18.2189
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0864	5.0218	0.0165	0.0596	-0.0401	25.4039
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.0864	5.0218	0.0165	0.0596	-0.0401	25.4039
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7936	-1.7249	-0.0109	-0.0388	0.0269	-20.4538
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4908	-1.2039	-0.0128	-0.0454	0.0331	-24.2851

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.8215
Shear X	-3.4811
Shear Z	0.0322
Moment X	0.1164
Moment Y (Twist)	0.0771
Moment Z	43.8013

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.2662
Shear X	-2.0864
Shear Z	0.0179
Moment X	0.0652
Moment Y (Twist)	0.0417
Moment Z	25.4039

### 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	-0.0007	2.3261	-0.0000	0.0000	0.0000	0.0287
ULS: 2. D + L	-0.0007	2.3261	-0.0000	0.0000	0.0000	0.0287
ULS: 3. D + (S or Lr or R)	-0.0028	7.9949	0.0000	0.0000	-0.0000	0.0694
ULS: 3. D + (S or Lr or R)	-0.0007	2.3261	-0.0000	0.0000	0.0000	0.0287
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0023	6.5777	0.0000	0.0000	-0.0000	0.0592
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0007	2.3261	-0.0000	0.0000	0.0000	0.0287
ULS: 5b. D + 0.7E	-0.0007	2.3261	-0.0000	0.0000	0.0000	0.0287
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0023	6.5777	0.0000	0.0000	-0.0000	0.0592
ULS: 8. 0.6D + 0.7E	-0.0004	1.3957	-0.0000	0.0000	0.0000	0.0172
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1173	6.0079	-0.0000	0.0000	0.0000	25.7995
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1173	6.0079	-0.0000	0.0000	0.0000	25.7995
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8146	-0.8298	-0.0000	0.0000	0.0000	-20.6933
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.5084	-0.3040	-0.0000	0.0000	0.0000	-24.5824
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5897	9.3390	0.0000	0.0000	-0.0000	19.3873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5897	9.3390	0.0000	0.0000	-0.0000	19.3873
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3592	4.2108	0.0000	0.0000	-0.0000	-15.4823
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1295	4.6051	0.0000	0.0000	-0.0000	-18.3991

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5881	5.0874	-0.0000	0.0000	0.0000	19.3568
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5881	5.0874	-0.0000	0.0000	0.0000	19.3568
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3608	-0.0408	-0.0000	0.0000	0.0000	-15.5128
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1311	0.3535	-0.0000	0.0000	0.0000	-18.4296
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1170	5.0774	0.0000	0.0000	0.0000	25.7880
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1170	5.0774	0.0000	0.0000	0.0000	25.7880
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8149	-1.7602	-0.0000	0.0000	0.0000	-20.7048
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5087	-1.2345	-0.0000	0.0000	0.0000	-24.5939

#### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	14.9294
Shear X	-3.5284
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0001
Moment Z	44.5475

#### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.3390
Shear X	-2.1173
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	25.7995

#### 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	0.0037	2.3154	-0.0019	-0.0070	0.0039	-0.0093
ULS: 2. D + L	0.0037	2.3154	-0.0019	-0.0070	0.0039	-0.0093
ULS: 3. D + (S or Lr or R)	0.0154	7.9505	-0.0078	-0.0291	0.0161	-0.0892
ULS: 3. D + (S or Lr or R)	0.0037	2.3154	-0.0019	-0.0070	0.0039	-0.0093
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0125	6.5417	-0.0063	-0.0236	0.0131	-0.0692
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0037	2.3154	-0.0019	-0.0070	0.0039	-0.0093
ULS: 5b. D + 0.7E	0.0037	2.3154	-0.0019	-0.0070	0.0039	-0.0093
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0125	6.5417	-0.0063	-0.0236	0.0131	-0.0692
ULS: 8. 0.6D + 0.7E	0.0022	1.3892	-0.0011	-0.0042	0.0023	-0.0056
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0849	5.9480	-0.0173	-0.0624	0.0417	25.4002
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0849	5.9480	-0.0173	-0.0624	0.0417	25.4002
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7951	-0.7988	0.0101	0.0360	-0.0254	-20.4576
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4923	-0.2778	0.0120	0.0427	-0.0315	-24.2888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5540	9.2662	-0.0179	-0.0651	0.0414	18.9879
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5540	9.2662	-0.0179	-0.0651	0.0414	18.9879
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3560	4.2062	0.0027	0.0086	-0.0089	-15.4054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1290	4.5969	0.0041	0.0136	-0.0135	-18.2788
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5628	5.0398	-0.0134	-0.0485	0.0322	19.0478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5628	5.0398	-0.0134	-0.0485	0.0322	19.0478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3473	-0.0202	0.0071	0.0252	-0.0181	-15.3455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1202	0.3705	0.0086	0.0303	-0.0227	-18.2189
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0864	5.0218	-0.0165	-0.0596	0.0401	25.4039
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.0864	5.0218	-0.0165	-0.0596	0.0401	25.4039
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7936	-1.7249	0.0109	0.0388	-0.0269	-20.4538
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4908	-1.2039	0.0128	0.0455	-0.0331	-24.2851

#### 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	14.8215
Shear X	-3.4811
Shear Z	-0.0322
Moment X	-0.1164
Moment Y (Twist)	0.0771
Moment Z	43.8014

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

Result	Value (kip, kip-ft)
Axial	9.2662
Shear X	-2.0864
Shear Z	-0.0179
Moment X	-0.0651
Moment Y (Twist)	0.0417
Moment Z	25.4039

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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0034	2.2072	0.0041	0.0135	-0.0202	0.0627
ULS: 2. D + L	-0.0034	2.2072	0.0041	0.0135	-0.0202	0.0627
ULS: 3. D + (S or Lr or R)	-0.0140	7.4967	0.0166	0.0552	-0.0837	0.2100
ULS: 3. D + (S or Lr or R)	-0.0034	2.2072	0.0041	0.0135	-0.0202	0.0627
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0114	6.1743	0.0135	0.0448	-0.0678	0.1732
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0034	2.2072	0.0041	0.0135	-0.0202	0.0627
ULS: 5b. D + 0.7E	-0.0034	2.2072	0.0041	0.0135	-0.0202	0.0627
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0114	6.1743	0.0135	0.0448	-0.0678	0.1732
ULS: 8. 0.6D + 0.7E	-0.0020	1.3243	0.0024	0.0081	-0.0121	0.0376
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0009	5.6443	0.0036	0.0125	-0.0110	24.3999
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.0009	5.6443	0.0036	0.0125	-0.0110	24.3999
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7072	-0.7383	0.0031	0.0096	-0.0247	-19.5509
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4281	-0.2492	0.0081	0.0275	-0.0360	-23.2970
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5095	8.7521	0.0131	0.0440	-0.0610	18.4261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5095	8.7521	0.0131	0.0440	-0.0610	18.4261
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2716	3.9652	0.0128	0.0418	-0.0712	-14.5370
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0623	4.3320	0.0165	0.0553	-0.0797	-17.3466
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5015	4.7850	0.0037	0.0128	-0.0133	18.3156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.5015	4.7850	0.0037	0.0128	-0.0133	18.3156
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2795	-0.0019	0.0033	0.0106	-0.0235	-14.6475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0702	0.3649	0.0071	0.0240	-0.0320	-17.4571
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9996	4.7614	0.0020	0.0071	-0.0030	24.3748
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9996	4.7614	0.0020	0.0071	-0.0030	24.3748
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7085	-1.6212	0.0015	0.0042	-0.0166	-19.5759
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4295	-1.1321	0.0065	0.0221	-0.0279	-23.3221

### 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	13.9765
Shear X	-3.3404
Shear Z	0.0303
Moment X	0.1016
Moment Y (Twist)	0.1437
Moment Z	42.1228

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.7521
Shear X	-2.0009
Shear Z	0.0166
Moment X	0.0553
Moment Y (Twist)	0.0837
Moment Z	24.3999

## Project Details

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

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

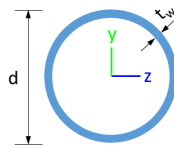


## Design Input Information

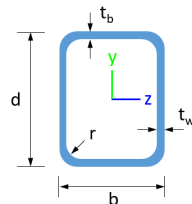
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

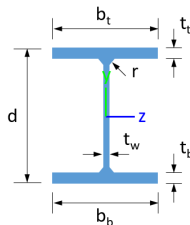
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
8	6in Pipe Sch 80	6.63	0.43				



ID	Name	d (in)	b (in)	$t_w$ (in)	$t_b$ (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

### 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> )
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60

16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K <sub>z</sub> L (ft)	K <sub>y</sub> L (ft)	L <sub>b</sub> (ft)	C <sub>b</sub>	L	S	T
1	8	24.70	24.70	11.76	-	3	2	1
2	5	4.20	4.20	2.00	-	3	2	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	3	2	1
4	16	2.44	2.44	3.75	1.69,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.86,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.58	3	2	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	3	2	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	3	2	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	3	2	1
8	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.12,2.11,2.37,2.12,2.12,2.11,1.40,2.12,2.12,2.12,2.15,2.12,2.12,2.11,2.35,2.11,2.11,2.11,1.40	3	2	1
9	2	2.60	2.60	4.00	-	3	2	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.86,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.58	3	2	1
11	19	1.33	1.33	2.05	2.12,2.12,2.12,2.12,2.12,2.12,2.24,2.24,2.30,2.33,2.28,2.28,2.38,2.37,2.15,2.15,2.11,2.09,2.24,2.24,2.33,2.34,2.29,2.29,2.37,2.37	3	2	1
12	5	1.30	1.30	2.00	-	3	2	1
13	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.07,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.10,1.09,1.09,1.08,1.09,1.09,1.09,1.09,1.09	3	2	1
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.16,1.09,1.09,1.09,1.32,1.09,1.09,1.10,1.10,1.09,1.09,1.09,1.19,1.09,1.09,1.09,1.03	3	2	1
15	19	9.97	9.97	4.75	2.33,2.33	3	2	1
16	19	9.97	9.97	4.75	2.33,2.33	3	2	1
101	8	24.70	24.70	11.76	-	3	2	1
102	5	1.30	1.30	2.00	-	3	2	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	3	2	1
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.85,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.70,1.67,1.67,1.66,1.58	3	2	1
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	3	2	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18	3	2	1
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	3	2	1





412	5	4.20	4.20	2.00	-	000	000	1
413	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.07,1.09,1.09,1.09,1.09,1.09,1.10,1.10,1.09,1.09,1.08,1.09,1.09,1.09,1.09	300	200	1
414	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.16,1.09,1.09,1.09,1.34,1.09,1.09,1.10,1.10,1.09,1.09,1.19,1.09,1.09,1.09,1.04	300	200	1
415	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.18,1.20,1.20,1.18,1.18,1.18,1.19,1.17,1.17,1.17,1.16,1.18,1.18,1.20,1.19,1.18,1.18,1.19	300	200	1
416	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.19,1.17,1.17,1.17,2.42,1.17,1.17,1.17,1.17,1.17,1.17,1.19,1.17,1.17,1.17,1.09	300	200	1

## Member Design Capacity

Member ID	$\Phi_c P_n$ (kip)	$\Phi_t P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	378.22	104.11	62.23	62.23	113.47	113.47
2	198.33	182.14	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	126.01	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.52	6.12	40.24	43.62
14	133.20	104.94	23.60	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62
101	378.22	104.11	62.23	62.23	113.47	113.47
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	182.14	21.95	21.95	59.50	59.50
113	133.20	104.94	23.83	6.12	40.24	43.62
114	133.20	104.94	23.83	6.12	40.24	43.62
115	133.20	69.16	17.33	6.12	40.24	43.62
116	133.20	69.16	17.80	6.12	40.24	43.62
201	378.22	104.11	62.23	62.23	113.47	113.47
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28

208	133.20	126.01	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	126.01	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	104.94	23.83	6.12	40.24	43.62
214	133.20	104.94	23.37	6.12	40.24	43.62
215	133.20	69.16	17.64	6.12	40.24	43.62
216	133.20	69.16	17.49	6.12	40.24	43.62
301	378.22	104.11	62.23	62.23	113.47	113.47
302	198.33	182.14	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	126.01	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	126.01	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	104.94	23.83	6.12	40.24	43.62
314	133.20	104.94	23.83	6.12	40.24	43.62
315	133.20	69.16	17.49	6.12	40.24	43.62
316	133.20	69.16	16.87	6.12	40.24	43.62
401	378.22	104.11	62.23	62.23	113.47	113.47
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28
408	133.20	32.95	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	32.95	32.87	6.12	40.24	43.62
412	198.33	182.14	21.95	21.95	59.50	59.50
413	133.20	104.94	24.52	6.12	40.24	43.62
414	133.20	104.94	23.83	6.12	40.24	43.62
415	133.20	69.16	17.95	6.12	40.24	43.62
416	133.20	69.16	16.87	6.12	40.24	43.62

## 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	δ	Status
1	0.134	0.677	0.004	0.029	0.000	0.730	#13	0.675	Not Required	Pass
2	0.004	0.521	0.165	0.114	0.028	0.657	#21	0.114	Not Required	Pass
3	0.011	0.752	0.060	0.075	0.006	0.817	#21	0.045	Not Required	Pass
4	0.011	0.746	0.196	0.075	0.040	0.872	#21	0.080	Not Required	Pass
5	0.011	0.466	0.205	0.074	0.051	0.517	#21	0.074	Not Required	Pass
6	0.012	0.745	0.068	0.074	0.007	0.818	#21	0.045	Not Required	Pass
7	0.012	0.463	0.211	0.074	0.054	0.521	#21	0.074	Not Required	Pass
8	0.001	0.055	0.182	0.052	0.020	0.225	#21	0.095	Not Required	Pass

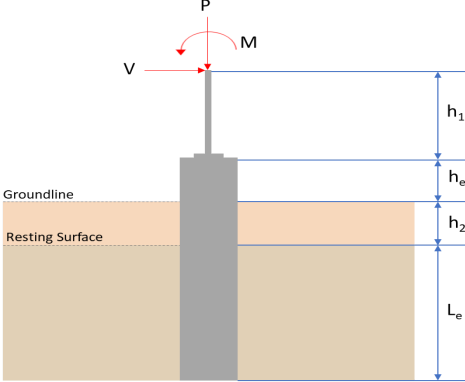
9	0.019	0.072	0.056	0.001	0.000	0.128	#21	0.204	Not Required	Pass
10	0.012	0.736	0.206	0.074	0.044	0.878	#21	0.080	Not Required	Pass
11	0.000	0.055	0.185	0.052	0.020	0.229	#21	0.095	Not Required	Pass
12	0.004	0.512	0.163	0.114	0.028	0.643	#21	0.035	Not Required	Pass
13	0.009	0.297	0.489	0.067	0.026	0.772	#21	0.286	Not Required	Pass
14	0.009	0.304	0.489	0.066	0.026	0.772	#21	0.190	Not Required	Pass
15	0.000	0.115	0.261	0.040	0.015	0.376	#21	Not Required	Not Required	Pass
16	0.000	0.115	0.261	0.040	0.015	0.376	#21	Not Required	Not Required	Pass
101	0.142	0.704	0.004	0.031	0.000	0.762	#13	0.675	Not Required	Pass
102	0.004	0.540	0.167	0.121	0.029	0.673	#21	0.035	Not Required	Pass
103	0.012	0.788	0.074	0.078	0.011	0.869	#21	0.045	Not Required	Pass
104	0.012	0.782	0.195	0.078	0.042	0.921	#21	0.080	Not Required	Pass
105	0.012	0.489	0.201	0.078	0.050	0.541	#21	0.074	Not Required	Pass
106	0.012	0.799	0.076	0.080	0.012	0.879	#21	0.045	Not Required	Pass
107	0.012	0.496	0.200	0.079	0.050	0.548	#21	0.074	Not Required	Pass
108	0.001	0.068	0.176	0.051	0.020	0.210	#21	0.095	Not Required	Pass
109	0.015	0.068	0.052	0.001	0.000	0.128	#21	0.204	Not Required	Pass
110	0.012	0.789	0.193	0.079	0.041	0.922	#21	0.080	Not Required	Pass
111	0.001	0.067	0.179	0.051	0.020	0.214	#21	0.095	Not Required	Pass
112	0.003	0.549	0.171	0.122	0.030	0.686	#21	0.114	Not Required	Pass
113	0.009	0.247	0.465	0.065	0.026	0.688	#21	0.286	Not Required	Pass
114	0.009	0.251	0.462	0.065	0.026	0.686	#21	0.286	Not Required	Pass
115	0.001	0.264	0.256	0.050	0.020	0.521	#21	0.473	Not Required	Pass
116	0.001	0.266	0.259	0.050	0.020	0.525	#21	0.473	Not Required	Pass
201	0.143	0.716	0.000	0.031	0.000	0.772	#13	0.675	Not Required	Pass
202	0.004	0.549	0.172	0.122	0.030	0.686	#21	0.035	Not Required	Pass
203	0.012	0.799	0.074	0.080	0.011	0.879	#21	0.045	Not Required	Pass
204	0.012	0.794	0.194	0.080	0.041	0.931	#21	0.080	Not Required	Pass
205	0.012	0.496	0.201	0.079	0.051	0.549	#21	0.074	Not Required	Pass
206	0.012	0.799	0.074	0.080	0.011	0.879	#21	0.045	Not Required	Pass
207	0.012	0.496	0.201	0.079	0.051	0.549	#21	0.074	Not Required	Pass
208	0.001	0.065	0.178	0.051	0.020	0.220	#21	0.095	Not Required	Pass
209	0.016	0.067	0.053	0.001	0.000	0.127	#21	0.204	Not Required	Pass
210	0.012	0.794	0.194	0.080	0.041	0.931	#21	0.080	Not Required	Pass
211	0.001	0.067	0.180	0.051	0.020	0.220	#21	0.095	Not Required	Pass
212	0.004	0.549	0.172	0.122	0.030	0.686	#21	0.035	Not Required	Pass
213	0.009	0.250	0.465	0.066	0.026	0.692	#21	0.286	Not Required	Pass
214	0.009	0.258	0.462	0.066	0.026	0.693	#21	0.286	Not Required	Pass
215	0.001	0.279	0.256	0.051	0.020	0.535	#21	0.473	Not Required	Pass
216	0.001	0.278	0.259	0.051	0.020	0.534	#21	0.473	Not Required	Pass
301	0.142	0.704	0.004	0.031	0.000	0.762	#13	0.675	Not Required	Pass
302	0.003	0.549	0.171	0.122	0.030	0.686	#21	0.114	Not Required	Pass
303	0.012	0.799	0.076	0.080	0.012	0.879	#21	0.045	Not Required	Pass
304	0.012	0.789	0.193	0.079	0.041	0.922	#21	0.080	Not Required	Pass
305	0.012	0.496	0.200	0.079	0.050	0.548	#21	0.074	Not Required	Pass
306	0.012	0.788	0.074	0.078	0.011	0.869	#21	0.045	Not Required	Pass
307	0.012	0.489	0.201	0.078	0.050	0.541	#21	0.074	Not Required	Pass
308	0.001	0.065	0.178	0.050	0.020	0.218	#21	0.095	Not Required	Pass
309	0.015	0.068	0.052	0.001	0.000	0.128	#21	0.204	Not Required	Pass
310	0.012	0.782	0.195	0.078	0.042	0.921	#21	0.080	Not Required	Pass
311	0.000	0.065	0.180	0.050	0.020	0.220	#21	0.095	Not Required	Pass
312	0.004	0.540	0.167	0.121	0.029	0.673	#21	0.035	Not Required	Pass
313	0.009	0.247	0.465	0.065	0.026	0.688	#21	0.286	Not Required	Pass
314	0.009	0.251	0.462	0.065	0.026	0.686	#21	0.286	Not Required	Pass

315	0.001	0.279	0.256	0.051	0.020	0.535	#21	0.473	Not Required	Pass
316	0.001	0.278	0.258	0.051	0.020	0.536	#21	0.473	Not Required	Pass
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405	0.012	0.463	0.211	0.074	0.054	0.521	#21	0.074	Not Required	Pass
406	0.011	0.752	0.060	0.075	0.006	0.817	#21	0.045	Not Required	Pass
407	0.011	0.466	0.205	0.074	0.051	0.517	#21	0.074	Not Required	Pass
408	0.000	0.115	0.261	0.040	0.015	0.376	#21	Not Required	Not Required	Pass
409	0.019	0.072	0.056	0.001	0.000	0.128	#21	0.204	Not Required	Pass
410	0.011	0.746	0.196	0.075	0.040	0.872	#21	0.080	Not Required	Pass
411	0.000	0.115	0.261	0.040	0.015	0.376	#21	Not Required	Not Required	Pass
412	0.004	0.521	0.165	0.114	0.028	0.657	#21	0.114	Not Required	Pass
413	0.009	0.297	0.489	0.067	0.026	0.772	#21	0.190	Not Required	Pass
414	0.009	0.304	0.489	0.066	0.026	0.772	#21	0.286	Not Required	Pass
415	0.001	0.261	0.257	0.052	0.020	0.518	#21	0.473	Not Required	Pass
416	0.001	0.262	0.259	0.052	0.020	0.521	#21	0.473	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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <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 resting 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 1102 1193 1191"> <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 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.752</td> <td>13.976</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.001</td> <td>-3.340</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.017</td> <td>-0.030</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.055</td> <td>-0.101</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>24.400</td> <td>42.122</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)	8.752	13.976	$V_x$ (kip)	-2.001	-3.340	$V_z$ (kip)	-0.017	-0.030	$M_x$ (kipft)	-0.055	-0.101	$M_z$ (kipft)	24.400	42.122	
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$M_z$ (kipft)	24.400	42.122																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.001 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.31863 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24.4 \text{ kipft}) + ((-2.001 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8854 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.84 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.002707 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((-0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.82733 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.84 \text{ ft}), (0.82733 \text{ ft})]$$

$$L_{e,req} = 5.84 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.84 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9344$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.752 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.547 \text{ kip/ft}^2$$

$$q = 0.547 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.547 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2735$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.31863 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.8854 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.8854 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8854 \text{ kipft/ft})) + (4 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2993 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8854 \text{ kipft/ft})) + (3 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.8854 \text{ kipft/ft})) + (2 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22903 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.8854 \text{ kipft/ft})) + ((-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.88769 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2993 \text{ ft})}{2}$$

$$p_a = 0.32245 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22903 \text{ kip/ft}^2)}{(0.32245 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.71027$$

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

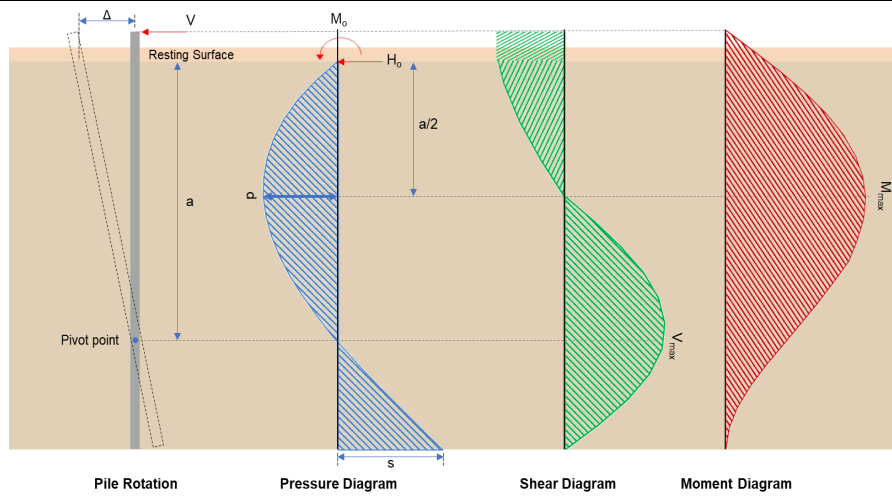
Status: **PASS**  
Ratio: **0.710**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.88769 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.94687$	Status: <b>PASS</b> Ratio: <b>0.950</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.002707 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.008758 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (-0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4599 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.008758 \text{ kipft/ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.008758 \text{ kipft/ft})) + (2 \times (-0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = -0.00062766 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.008758 \text{ kipft/ft})) + ((-0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.00009172 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4599 \text{ ft})}{2}$ $p_a = 0.33449 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.00062766 \text{ kip/ft}^2)}{(0.33449 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0018765$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.000</b>

$$\text{Ratio} = \frac{(0.00009172 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.000097834$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.34 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.53185 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(42.122 \text{ kipft}) + ((-3.34 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7073 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.7073 \text{ kipft/ft})}{(-0.53185 \text{ kip/ft})}$$

$$E = 12.611 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.7073 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.53185 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.7073 \text{ kipft/ft})) + (4 \times (-0.53185 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.296 \text{ ft}$$

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

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

$$V_{max} = ((-0.53185 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.611 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.296 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.611 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.296 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.0006 \text{ kip}$$

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

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

$$M_{max} = ((-0.53185 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(12.611 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.611 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (12.611 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.917 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0047771 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.101 \text{ kipft}) + ((-0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016083 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.016083 \text{ kipft/ft})}{(-0.0047771 \text{ kip/ft})}$$

$$E = 3.3667 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.016083 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.016083 \text{ kipft/ft})) + (4 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4547 \text{ ft}$$

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

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

$$V_{max} = ((-0.0047771 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.3667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4547 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.3667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4547 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.03093 \text{ kip}$$

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

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

$$M_{max} = ((-0.0047771 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.3667 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4547 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.3667 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4547 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.3667 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4547 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.085995 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.976 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.132 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.132 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.976 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0052243$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.976 \text{ kip} \rightarrow 13976 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13976 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.35 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

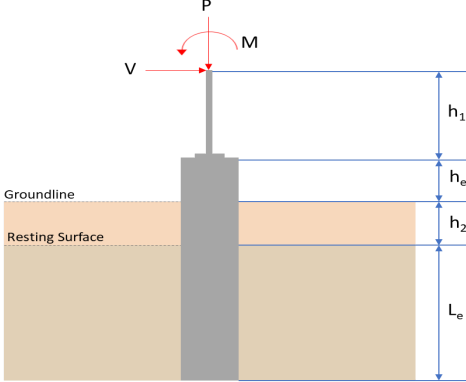
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.35 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.35 \text{ kip}$$

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.31 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.0006 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.0006 \text{ kip})}{(111.31 \text{ kip})}$ $\text{Ratio} = 0.080862$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.03093 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.03093 \text{ kip})}{(111.31 \text{ kip})}$ $\text{Ratio} = 0.00027788$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</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> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 26.917 \text{kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(26.917 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.10784$	<p>Status: <b>PASS</b>          Ratio: <b>0.110</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.085995 \text{kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.085995 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00034453$	<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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <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 resting 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 1102 1193 1191"> <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 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>9.266</td> <td>14.821</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.086</td> <td>-3.481</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.018</td> <td>0.032</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.065</td> <td>0.116</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>25.404</td> <td>43.801</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)	9.266	14.821	$V_x$ (kip)	-2.086	-3.481	$V_z$ (kip)	0.018	0.032	$M_x$ (kipft)	0.065	0.116	$M_z$ (kipft)	25.404	43.801	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.086 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.33217 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.404 \text{ kipft}) + ((-2.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0452 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9052 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0028662 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.065 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.01035 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0001 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.9052 \text{ ft}), (1.0001 \text{ ft})]$$

$$L_{e,req} = 5.905 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.905 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9448$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.266 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.579125 \text{ kip/ft}^2$$

$$q = 0.01913 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.57913 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28956$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.33217 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0452 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.0452 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0452 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2994 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0452 \text{ kipft/ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0452 \text{ kipft/ft})) + (2 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.23823 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.0452 \text{ kipft/ft})) + ((-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.92381 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2994 \text{ ft})}{2}$$

$$p_a = 0.32246 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23823 \text{ kip/ft}^2)}{(0.32246 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73879$$

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

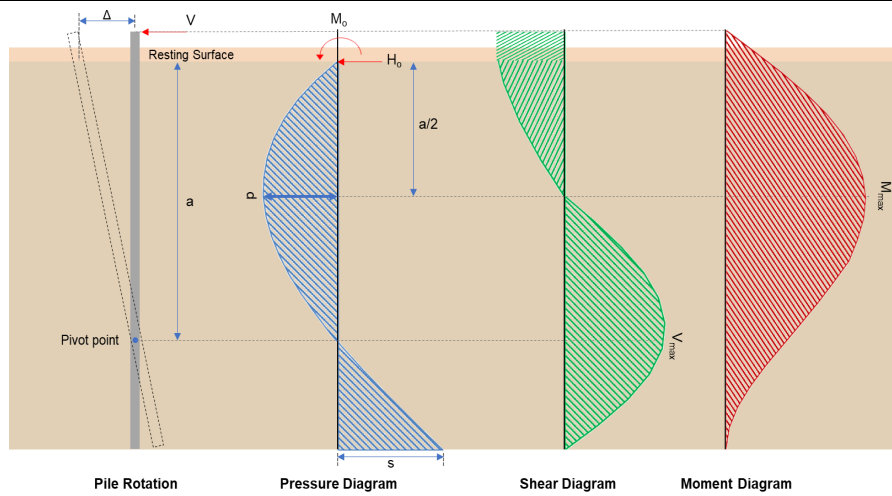
Status: **PASS**  
Ratio: **0.740**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.92381 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9854$	<p>Status: <b>PASS</b> Ratio: <b>0.990</b></p>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.0028662 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.01035 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.01035 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.01035 \text{ kipft/ft})) + (4 \times (0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4457 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.01035 \text{ kipft/ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.01035 \text{ kipft/ft})) + (2 \times (0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.0025988 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.01035 \text{ kipft/ft})) + ((0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.0059312 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4457 \text{ ft})}{2}$ $p_a = 0.33343 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0025988 \text{ kip/ft}^2)}{(0.33343 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0077941$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>

$$\text{Ratio} = \frac{(0.0059312 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0063266$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.481 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.5543 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(43.801 \text{ kipft}) + ((-3.481 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.9747 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.9747 \text{ kipft/ft})}{(-0.5543 \text{ kip/ft})}$$

$$E = 12.583 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.9747 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.5543 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.9747 \text{ kipft/ft})) + (4 \times (-0.5543 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2962 \text{ ft}$$

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

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

$$V_{max} = ((-0.5543 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2962 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2962 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.3626 \text{ kip}$$

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

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

$$M_{max} = ((-0.5543 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(12.583 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 27.998 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.032 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0050955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.116 \text{ kipft}) + ((0.032 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018471 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.018471 \text{ kipft/ft})}{(0.0050955 \text{ kip/ft})}$$

$$E = 3.625 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.018471 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0050955 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.018471 \text{ kipft/ft})) + (4 \times (0.0050955 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4452 \text{ ft}$$

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

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

$$V_{max} = ((0.0050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4452 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4452 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.034469 \text{ kip}$$

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

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

$$M_{max} = ((0.0050955 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.625 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.096328 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(14.821 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.104 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.104 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.821 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0055402$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 14.821 \text{ kip} \rightarrow 14821 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14821 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.46 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

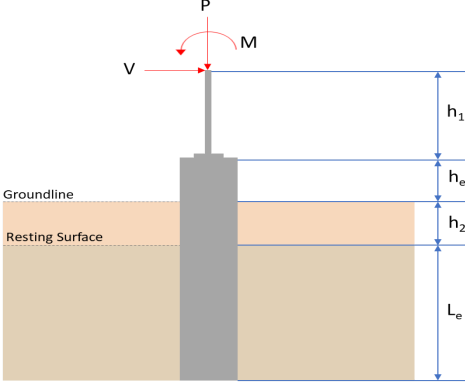
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.46 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.46 \text{ kip}$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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> $V_{s,b} = \frac{2 A_v f_{ytck} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.46 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.38 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.3626 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.3626 \text{ kip})}{(111.38 \text{ kip})}$ $\text{Ratio} = 0.084059$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.034469 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.034469 \text{ kip})}{(111.38 \text{ kip})}$ $\text{Ratio} = 0.00030947$	<p>Status: <b>PASS</b> Ratio: <b>0.080</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> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 27.998 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(27.998 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11217$	<p>Status: <b>PASS</b>  Ratio: <b>0.110</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.096328 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.096328 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00038593$	<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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <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 resting 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 1102 1193 1191"> <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 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>9.266</td> <td>14.821</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.086</td> <td>-3.481</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.018</td> <td>-0.032</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.065</td> <td>-0.116</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>25.404</td> <td>43.801</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)	9.266	14.821	$V_x$ (kip)	-2.086	-3.481	$V_z$ (kip)	-0.018	-0.032	$M_x$ (kipft)	-0.065	-0.116	$M_z$ (kipft)	25.404	43.801	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.086 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.33217 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.404 \text{ kipft}) + ((-2.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0452 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9052 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0028662 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.065 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.01035 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.87799 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.9052 \text{ ft}), (0.87799 \text{ ft})]$$

$$L_{e,req} = 5.905 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.905 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9448$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.266 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.579125 \text{ kip/ft}^2$$

$$q = 0.01913 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.57913 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28956$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.33217 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0452 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.0452 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0452 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2994 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0452 \text{ kipft/ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0452 \text{ kipft/ft})) + (2 \times (-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.23823 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.0452 \text{ kipft/ft})) + ((-0.33217 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.92381 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2994 \text{ ft})}{2}$$

$$p_a = 0.32246 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23823 \text{ kip/ft}^2)}{(0.32246 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73879$$

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

Status: **PASS**  
Ratio: **0.740**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.92381 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9854$$

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

**Considering z-direction:**

$H_o = -0.0028662 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.01035 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.01035 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.01035 \text{ kipft/ft})) + (4 \times (-0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4457 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.01035 \text{ kipft/ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.01035 \text{ kipft/ft})) + (2 \times (-0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0006121 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.01035 \text{ kipft/ft})) + ((-0.0028662 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.00042803 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4457 \text{ ft})}{2}$$

$$p_a = 0.33343 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0006121 \text{ kip/ft}^2)}{(0.33343 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0018358$$

$p_s$  - 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

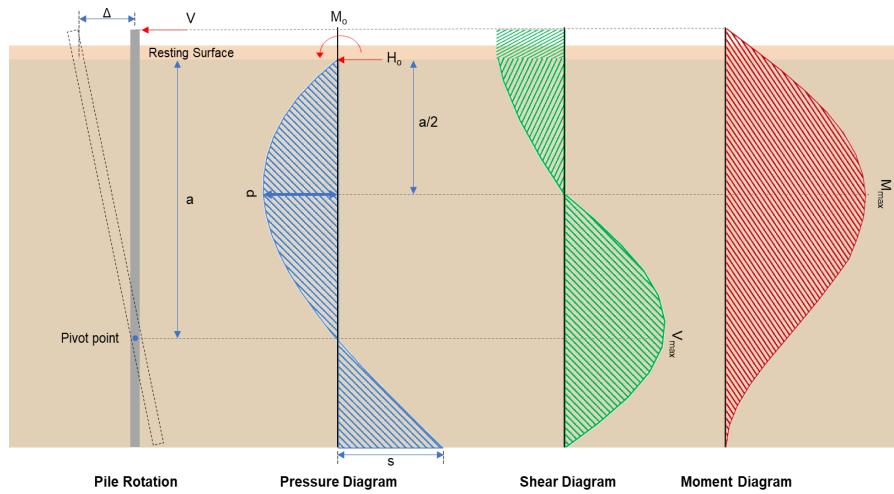
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.00042803 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00045656$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.481 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.5543 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(43.801 \text{ kipft}) + ((-3.481 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.9747 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.9747 \text{ kipft/ft})}{(-0.5543 \text{ kip/ft})}$$

$$E = 12.583 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.9747 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.5543 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.9747 \text{ kipft/ft})) + (4 \times (-0.5543 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2962 \text{ ft}$$

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

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

$$V_{max} = ((-0.5543 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2962 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2962 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.3626 \text{ kip}$$

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

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

$$M_{max} = ((-0.5543 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(12.583 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (12.583 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2962 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 27.998 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.032 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0050955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.116 \text{ kipft}) + ((-0.032 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.018471 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.018471 \text{ kipft/ft})}{(-0.0050955 \text{ kip/ft})}$$

$$E = 3.625 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.018471 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0050955 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.018471 \text{ kipft/ft})) + (4 \times (-0.0050955 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4452 \text{ ft}$$

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

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

$$V_{max} = ((-0.0050955 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4452 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4452 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.034469 \text{ kip}$$

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

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

$$M_{max} = ((-0.0050955 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.625 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (3.625 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4452 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.096328 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(14.821 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.104 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.104 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

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

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.821 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0055402$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 14.821 \text{ kip} \rightarrow 14821 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14821 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.46 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

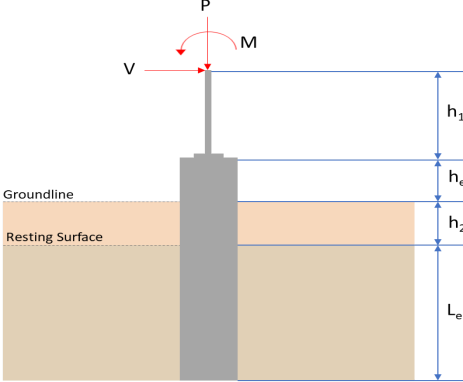
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.46 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.46 \text{ kip}$$

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ytik} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.46 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.38 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.3626 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.3626 \text{ kip})}{(111.38 \text{ kip})}$ $\text{Ratio} = 0.084059$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.034469 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.034469 \text{ kip})}{(111.38 \text{ kip})}$ $\text{Ratio} = 0.00030947$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</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> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 27.998 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(27.998 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11217$	<p>Status: <b>PASS</b>  Ratio: <b>0.110</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.096328 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.096328 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00038593$	<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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <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 resting 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 1102 1192 1191"> <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 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.752</td> <td>13.976</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.001</td> <td>-3.340</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.017</td> <td>0.030</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.055</td> <td>0.102</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>24.400</td> <td>42.123</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)	8.752	13.976	$V_x$ (kip)	-2.001	-3.340	$V_z$ (kip)	0.017	0.030	$M_x$ (kipft)	0.055	0.102	$M_z$ (kipft)	24.400	42.123	
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$M_z$ (kipft)	24.400	42.123																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.001 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.31863 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24.4 \text{ kipft}) + ((-2.001 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8854 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.84 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.002707 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.94917 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.84 \text{ ft}), (0.94917 \text{ ft})]$$

$$L_{e,req} = 5.84 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.84 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9344$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.752 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.547 \text{ kip/ft}^2$$

$$q = 0.041 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$Ratio = \frac{(0.547 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.2735$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.31863 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.8854 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.8854 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8854 \text{ kipft/ft})) + (4 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2993 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8854 \text{ kipft/ft})) + (3 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.8854 \text{ kipft/ft})) + (2 \times (-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22903 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.8854 \text{ kipft/ft})) + ((-0.31863 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.88769 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2993 \text{ ft})}{2}$$

$$p_a = 0.32245 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22903 \text{ kip/ft}^2)}{(0.32245 \text{ kip/ft}^2)}$$

$$Ratio = 0.71027$$

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

Status: **PASS**  
Ratio: **0.710**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.88769 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94687$$

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

**Considering z-direction:**

$H_o = 0.002707 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.008758 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4599 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.008758 \text{ kipft/ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.008758 \text{ kipft/ft})) + (2 \times (0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0023507 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.008758 \text{ kipft/ft})) + ((0.002707 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0052892 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4599 \text{ ft})}{2}$$

$$p_a = 0.33449 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0023507 \text{ kip/ft}^2)}{(0.33449 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0070278$$

$p_s$  - 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

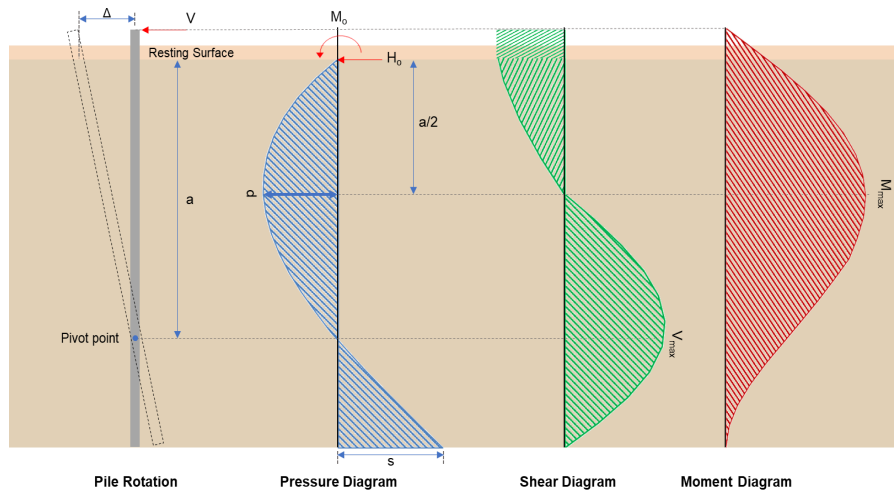
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0052892 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0056418$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.34 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.53185 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(42.123 \text{ kipft}) + ((-3.34 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7075 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.7075 \text{ kipft/ft})}{(-0.53185 \text{ kip/ft})}$$

$$E = 12.612 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.7075 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.53185 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.7075 \text{ kipft/ft})) + (4 \times (-0.53185 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.296 \text{ ft}$$

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

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

$$V_{max} = ((-0.53185 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.612 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.296 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.612 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.296 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.0007 \text{ kip}$$

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

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

$$M_{max} = ((-0.53185 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(12.612 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.612 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (12.612 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.296 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.918 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0047771 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.102 \text{ kipft}) + ((0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016242 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.016242 \text{ kipft/ft})}{(0.0047771 \text{ kip/ft})}$$

$$E = 3.4 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.016242 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.016242 \text{ kipft/ft})) + (4 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4535 \text{ ft}$$

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

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

$$V_{max} = ((0.0047771 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.4 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4535 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.4 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4535 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.031109 \text{ kip}$$

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

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

$$M_{max} = ((0.0047771 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(3.4 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4535 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.4 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4535 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.4 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4535 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.086551 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.976 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.132 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.132 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.976 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0052243$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.976 \text{ kip} \rightarrow 13976 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13976 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.35 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

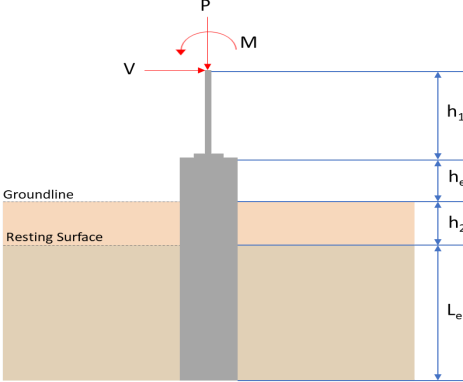
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.35 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.35 \text{ kip}$$

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.31 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.0007 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.0007 \text{ kip})}{(111.31 \text{ kip})}$ $\text{Ratio} = 0.080864$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.031109 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.031109 \text{ kip})}{(111.31 \text{ kip})}$ $\text{Ratio} = 0.00027949$	<p>Status: <b>PASS</b> Ratio: <b>0.080</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> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 26.918 \text{kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(26.918 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.10784$	<p>Status: <b>PASS</b>          Ratio: <b>0.110</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.086551 \text{kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.086551 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00034676$	<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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6.5</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 resting 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 1102 1193 1191"> <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 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>9.339</td> <td>14.929</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.117</td> <td>-3.528</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>25.799</td> <td>44.548</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)	9.339	14.929	$V_x$ (kip)	-2.117	-3.528	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	25.799	44.548	
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$M_z$ (kipft)	25.799	44.548																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.117 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.3371 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.799 \text{ kipft}) + ((-2.117 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1081 \text{ kipft/ft}$$

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:

$$L_{e,x} = 5.9314 \text{ ft} - \text{Required depth in x-direction,}$$

**Considering z-direction:**

$$L_{e,z} = 0 \text{ ft} - \text{Required depth in z-direction,}$$

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.9314 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.931 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.931 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.91246$$

Status: **PASS**  
Ratio: **0.910**

### End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.339 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58369 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

**Ratio** - Capacity

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

$$\text{Ratio} = \frac{(0.58369 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29184$$

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

Czerniak

### Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.3371$  kip/ft - Lateral force per length of pile,

$M_o = 4.1081$  kipft/ft - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.1081 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.3371 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.1081 \text{ kipft/ft})) + (4 \times (-0.3371 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4754 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1081 \text{ kipft/ft})) + (3 \times (-0.3371 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (4.1081 \text{ kipft/ft})) + (2 \times (-0.3371 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21725 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.1081 \text{ kipft/ft})) + ((-0.3371 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.85563 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4754 \text{ ft})}{2}$$

$$p_a = 0.33566 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21725 \text{ kip/ft}^2)}{(0.33566 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64725$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

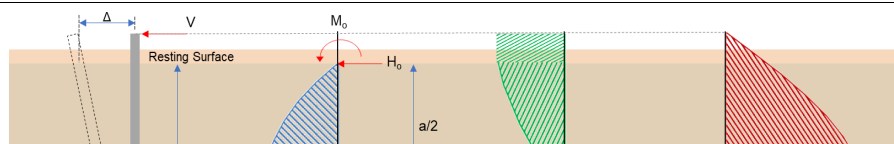
$$\text{Ratio} = \frac{s}{p_s}$$

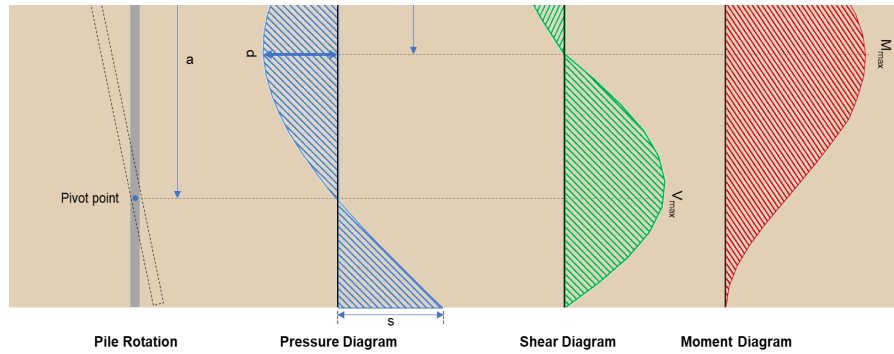
$$\text{Ratio} = \frac{(0.85563 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87757$$

Status: **PASS**  
Ratio: **0.650**

Status: **PASS**  
Ratio: **0.880**





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.528 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.56178 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(44.548 \text{ kipft}) + ((-3.528 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0936 \text{ kipft/ft}$$

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

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

$$E = \frac{(7.0936 \text{ kipft/ft})}{(-0.56178 \text{ kip/ft})}$$

$$E = 12.627 \text{ ft}$$

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

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

$$a = \frac{(4 \times (7.0936 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.56178 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.0936 \text{ kipft/ft})) + (4 \times (-0.56178 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4717 \text{ ft}$$

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

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

$$V_{max} = ((-0.56178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.627 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4717 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.627 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4717 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2077 \text{ kip}$$

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

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

$$M_{max} = ((-0.56178 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(12.627 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4717 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.627 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4717 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (12.627 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4717 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.597 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(14.929 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

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

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

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

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

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

**Axial Compression Strength (ACI 318-19, LRFD)**22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.929 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0055806$$

Status: **PASS**  
Ratio: **0.010****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$  - Effective width,22.5.2.2  $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 14.929 \text{ kip} \rightarrow 14929 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14929 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

 $V_c$  - Governing shear strength of concrete

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

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

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

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <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>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.48 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.39 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.2077 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.2077 \text{ kip})}{(111.39 \text{ kip})}$ $Ratio = 0.082661$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</b></p>
<p>14.5.2.1b</p>	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:</p> <p><math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$$\phi M_{n,2} = \phi S_x F_y$$

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 28.597 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(28.597 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.11457$$

Status: **PASS**  
Ratio: **0.110**