

# Your Project Calculations



Project Name: Tavoli Apartments-cuV2

S3D Model Link:  
[https://platform.skyciv.com/structural?preload\\_name=Tavoli%20Apartments-cuV2&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/5\\_2024](https://platform.skyciv.com/structural?preload_name=Tavoli%20Apartments-cuV2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024)

Public Model Link:  
[https://platform.skyciv.com/structural-viewer?project\\_id=XHVflip78k6ewwFIKcYn70uOX8mdQFcpTURzjhiPk3DcXcigdbLdMLmMoeE4VKE](https://platform.skyciv.com/structural-viewer?project_id=XHVflip78k6ewwFIKcYn70uOX8mdQFcpTURzjhiPk3DcXcigdbLdMLmMoeE4VKE)

## Array Specification

Product:	Beam
Unique ID:	3P-19.75-8TOP-HD-72-L-5Hx8W-F91F
Duty Classification:	HD
Module Width:	44.65 in
Module Length:	88.81in
Number of Rows:	5
Number of Columns:	8
Total Number of Modules:	40
Desired Tilt Angle:	10
Front Edge Clearance:	14
Total Array Height at Tilt:	17.25 ft
Total Frame Length:	59.00 ft
Frame Weight:	3164 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	59.87 ft
Rail Length:	225.75 in
Rail Spacing:	3.70 ft
Rail Check:	Not Checked

## Support Specifications

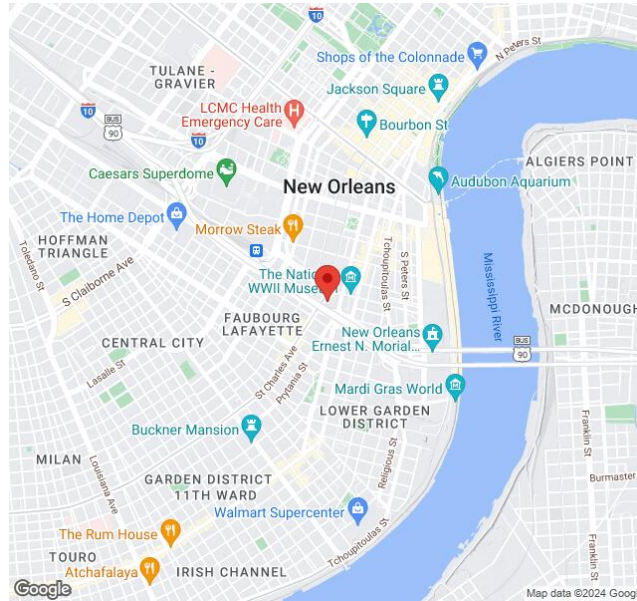
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	15.63 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

## Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft Pile 3: 6.25 ft
Foundation Volume:	11.111 y <sup>3</sup>
Foundation Result:	PASSED
Mount Twist:	0.218156 kip

## Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	1040 St Charles Ave, New Orleans, LA 70130, USA
Wind Speed:	134 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 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.

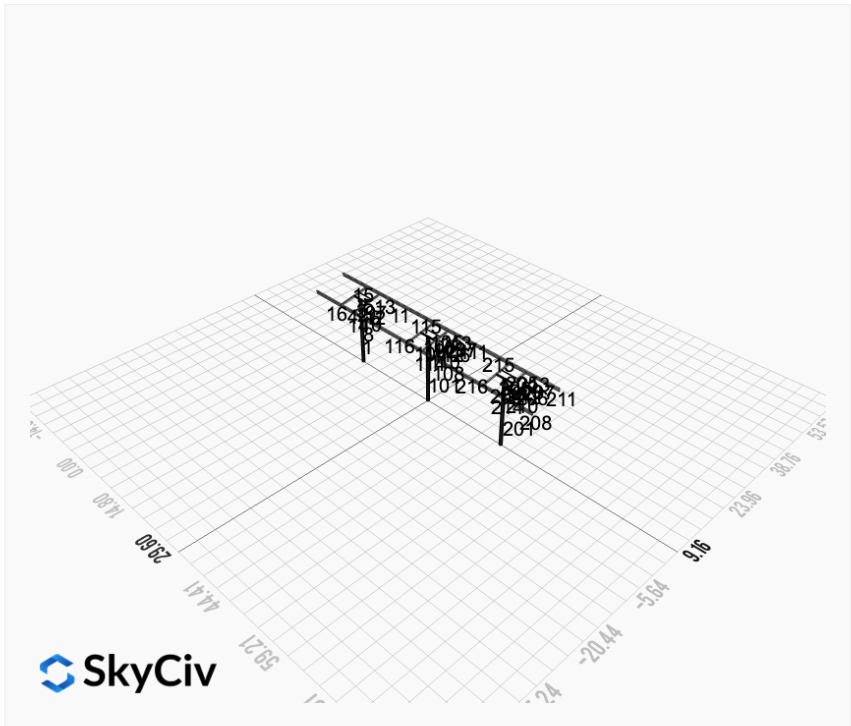
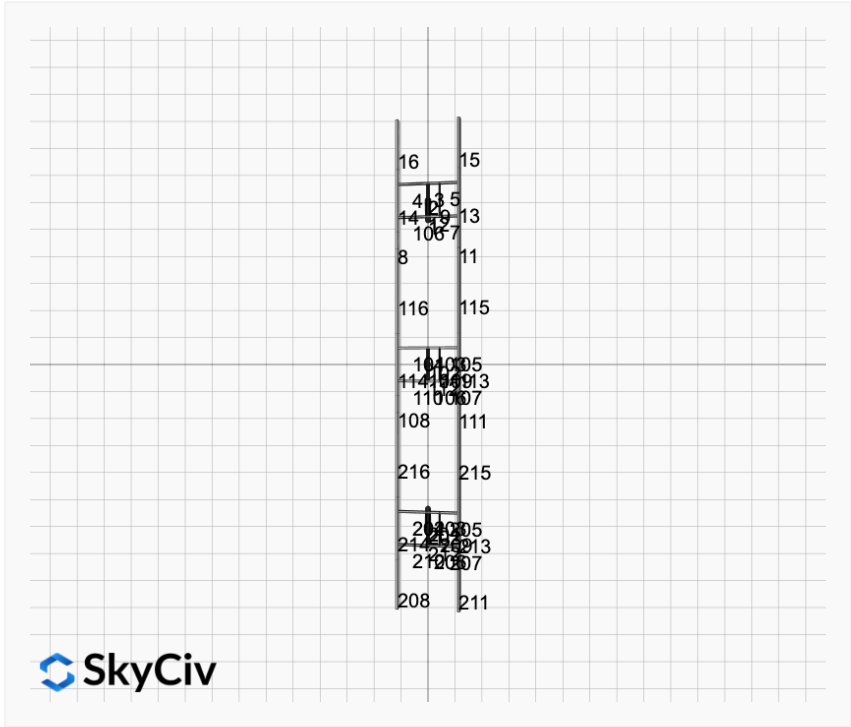
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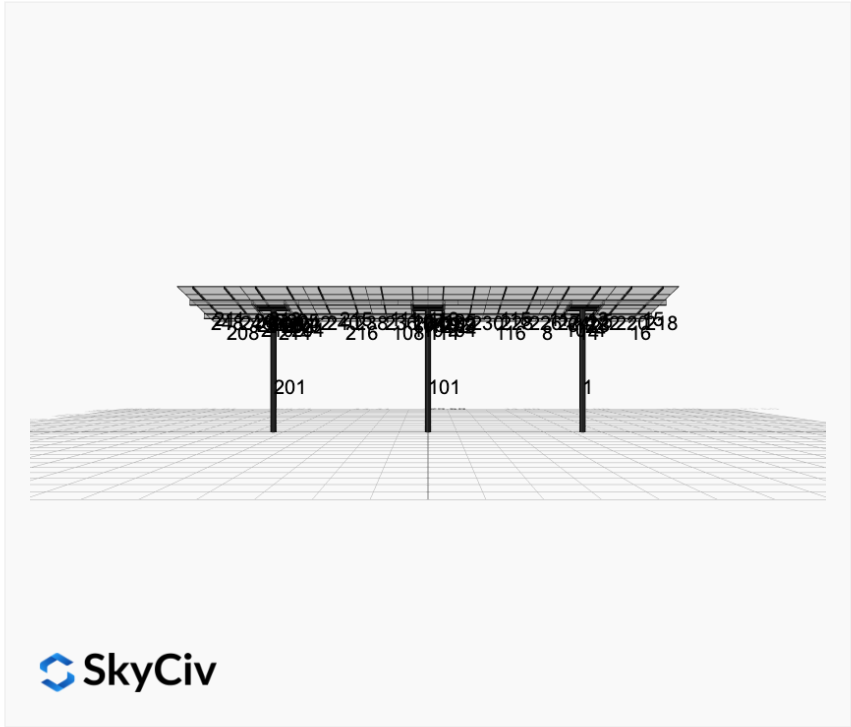
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### Design Notes:

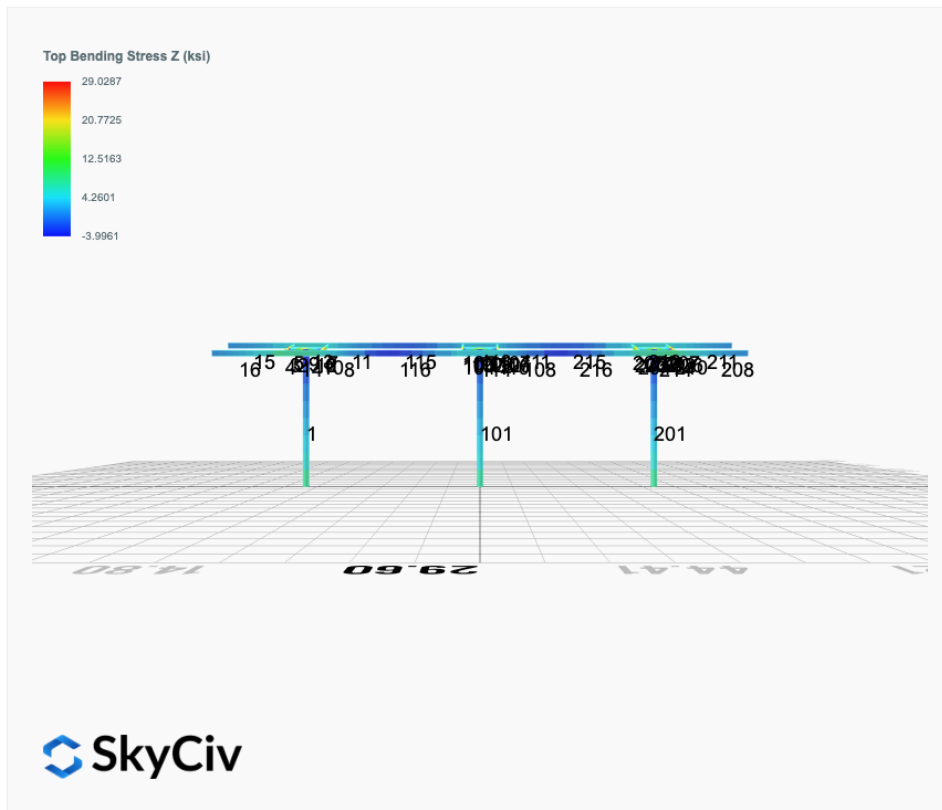
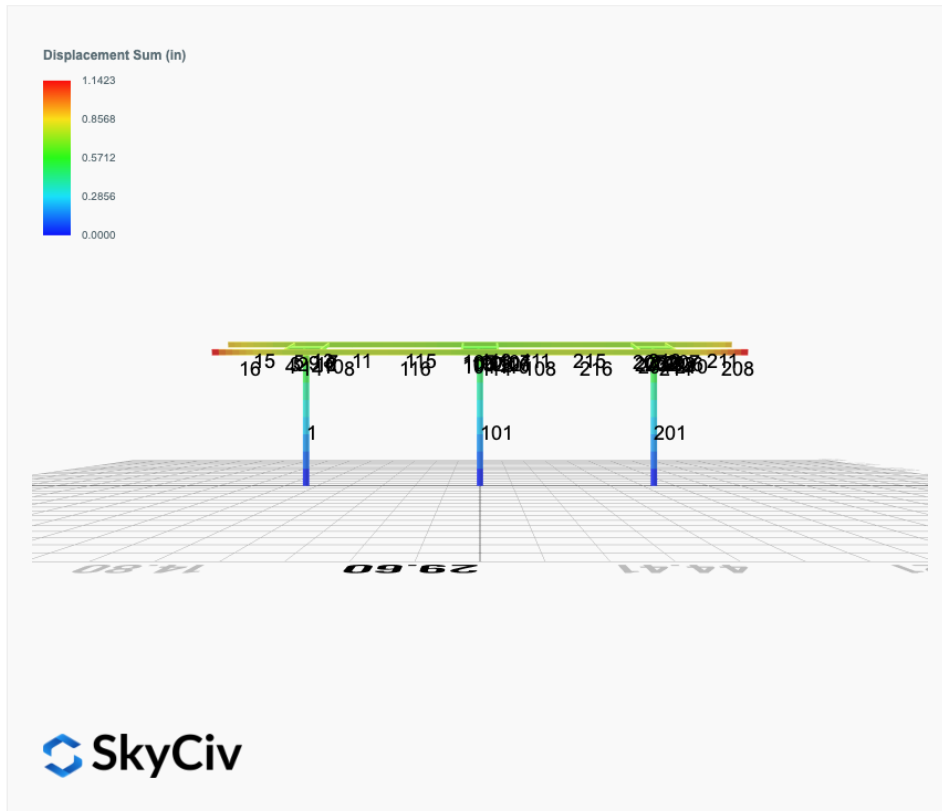
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

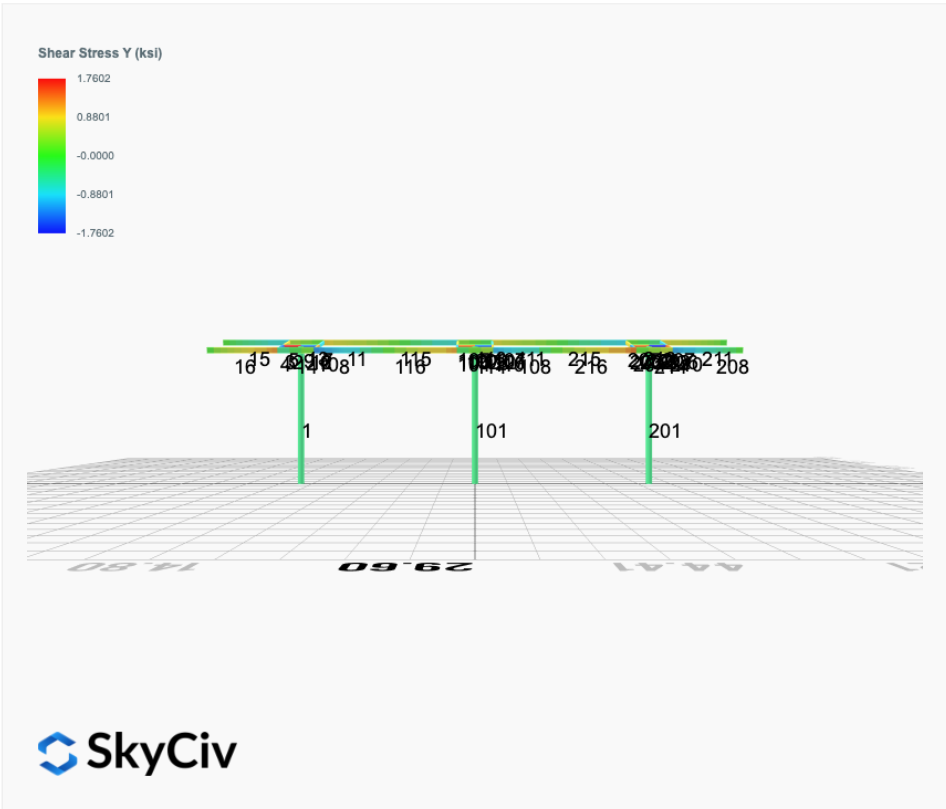
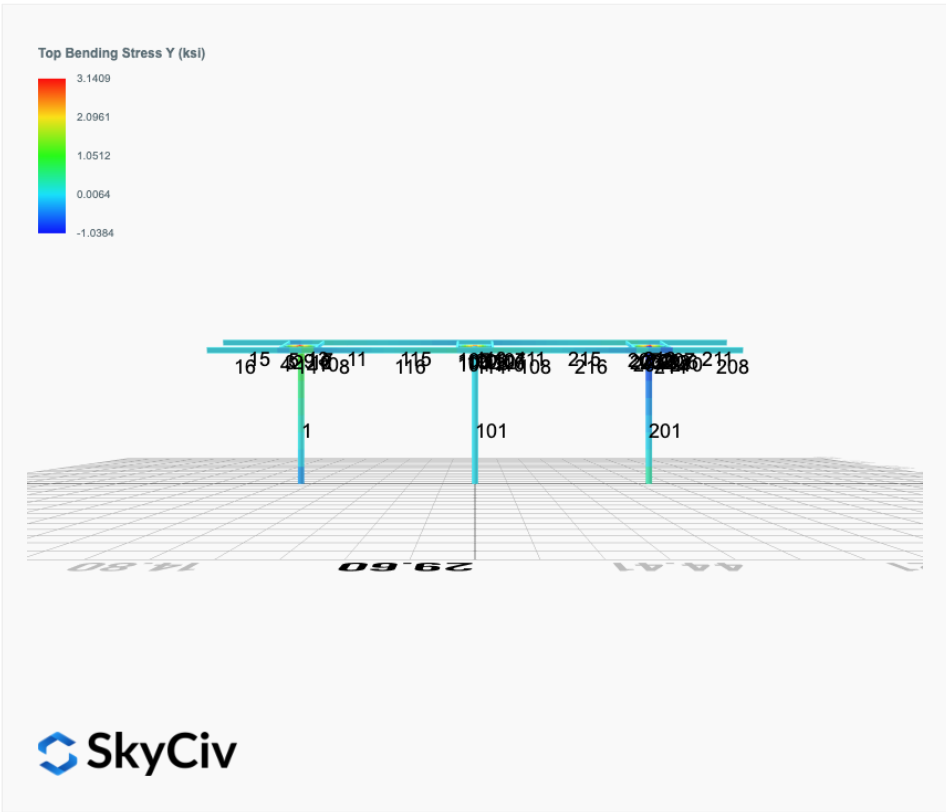


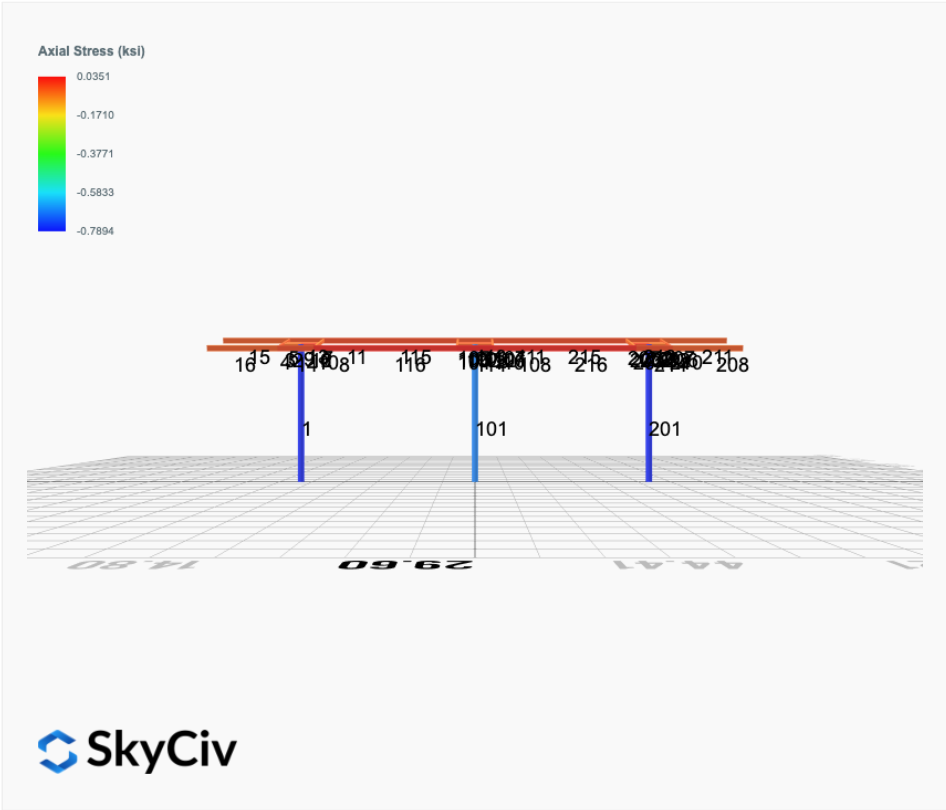




## FEM Results (Envelope Worst Case for each member)







## 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	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 2. D + L	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 3. D + (S or Lr or R)	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 3. D + (S or Lr or R)	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 5b. D + 0.7E	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0081	2.7792	-0.0433	-0.2137	0.0300	0.1401
ULS: 8. 0.6D + 0.7E	-0.0049	1.6675	-0.0260	-0.1282	0.0180	0.0840
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9940	8.4477	-0.1635	-0.8074	0.1387	20.8768
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.9940	8.4477	-0.1635	-0.8074	0.1387	20.8768
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6892	-1.1991	0.0379	0.1845	-0.0405	-7.1953
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5877	-0.6773	0.0324	0.1572	-0.0421	-21.2952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7475	7.0306	-0.1335	-0.6590	0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7475	7.0306	-0.1335	-0.6590	0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5149	-0.2045	0.0176	0.0849	-0.0229	-5.3614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4388	0.1868	0.0135	0.0645	-0.0241	-15.9364
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7475	7.0306	-0.1335	-0.6590	0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7475	7.0306	-0.1335	-0.6590	0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5149	-0.2045	0.0176	0.0849	-0.0229	-5.3614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4388	0.1868	0.0135	0.0645	-0.0241	-15.9364
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9908	7.3360	-0.1462	-0.7219	0.1267	20.8208
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.9908	7.3360	-0.1462	-0.7219	0.1267	20.8208
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6925	-2.3107	0.0552	0.2700	-0.0525	-7.2513
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5910	-1.7890	0.0497	0.2427	-0.0541	-21.3512

### 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	12.7822
Shear X	-1.6519
Shear Z	-0.2530
Moment X	-1.2521
Moment Y (Twist)	0.2181
Moment Z	36.3951

### 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	8.4477
Shear X	-0.9940
Shear Z	-0.1635
Moment X	-0.8074
Moment Y (Twist)	0.1387
Moment Z	21.3512

## 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	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 2. D + L	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 3. D + (S or Lr or R)	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 3. D + (S or Lr or R)	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 5b. D + 0.7E	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0162	2.5914	-0.0000	0.0000	-0.0000	-0.1911
ULS: 8. 0.6D + 0.7E	0.0097	1.5548	-0.0000	0.0000	-0.0000	-0.1147
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9202	7.7477	-0.0000	0.0000	-0.0000	19.5180
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.9202	7.7477	-0.0000	0.0000	-0.0000	19.5180
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6611	-1.0188	-0.0000	0.0000	-0.0000	-7.0243
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5997	-0.5629	-0.0000	0.0000	-0.0000	-20.8431
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6861	6.4586	-0.0000	0.0000	-0.0000	14.5908
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6861	6.4586	-0.0000	0.0000	-0.0000	14.5908
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4999	-0.1163	-0.0000	0.0000	-0.0000	-5.3160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4538	0.2257	-0.0000	0.0000	-0.0000	-15.6801
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6861	6.4586	-0.0000	0.0000	-0.0000	14.5908
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6861	6.4586	-0.0000	0.0000	-0.0000	14.5908
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4999	-0.1163	-0.0000	0.0000	-0.0000	-5.3160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4538	0.2257	-0.0000	0.0000	-0.0000	-15.6801
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9267	6.7111	-0.0000	0.0000	-0.0000	19.5945
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.9267	6.7111	-0.0000	0.0000	-0.0000	19.5945
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6546	-2.0554	-0.0000	0.0000	-0.0000	-6.9478
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5932	-1.5994	-0.0000	0.0000	-0.0000	-20.7667

#### 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	11.7041
Shear X	-1.5607
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0000
Moment Z	35.4439

#### 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	7.7477
Shear X	-0.9267
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	20.8431

#### 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	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 2. D + L	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 3. D + (S or Lr or R)	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 3. D + (S or Lr or R)	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 5b. D + 0.7E	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0081	2.7792	0.0433	0.2137	-0.0300	0.1401
ULS: 8. 0.6D + 0.7E	-0.0049	1.6675	0.0260	0.1282	-0.0180	0.0840
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9940	8.4477	0.1635	0.8074	-0.1387	20.8768
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.9940	8.4477	0.1635	0.8074	-0.1387	20.8768
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6892	-1.1991	-0.0379	-0.1845	0.0405	-7.1953
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5877	-0.6773	-0.0324	-0.1572	0.0421	-21.2952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7475	7.0306	0.1335	0.6590	-0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7475	7.0306	0.1335	0.6590	-0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5149	-0.2045	-0.0176	-0.0849	0.0229	-5.3614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4388	0.1868	-0.0135	-0.0645	0.0241	-15.9364

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7475	7.0306	0.1335	0.6590	-0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7475	7.0306	0.1335	0.6590	-0.1115	15.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5149	-0.2045	-0.0176	-0.0849	0.0229	-5.3614
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4388	0.1868	-0.0135	-0.0645	0.0241	-15.9364
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9908	7.3360	0.1462	0.7219	-0.1267	20.8208
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.9908	7.3360	0.1462	0.7219	-0.1267	20.8208
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6925	-2.3107	-0.0552	-0.2700	0.0525	-7.2513
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5910	-1.7890	-0.0497	-0.2427	0.0541	-21.3512

### 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	12.7822
Shear X	-1.6519
Shear Z	0.2530
Moment X	1.2521
Moment Y (Twist)	0.2182
Moment Z	36.3955

### 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	8.4477
Shear X	-0.9940
Shear Z	0.1635
Moment X	0.8074
Moment Y (Twist)	0.1387
Moment Z	21.3512

## Project Details

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

User Name: sales@mtsolar.us  
 Project Name: Tavoli Apartments-cuV2  
 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 <sub>y</sub> (ksi)	F <sub>u</sub> (ksi)
1	29000	50	65

Section Dimensions							

ID	Name	d (in)	t <sub>w</sub> (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
9	8in Pipe Sch 40	8.63	0.32				

ID	Name	d (in)	b (in)	t <sub>w</sub> (in)	t <sub>b</sub> (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	

ID	Name	d (in)	t <sub>w</sub> (in)	b <sub>t</sub> (in)	b <sub>b</sub> (in)	t <sub>t</sub> (in)	t <sub>b</sub> (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 <sub>yp</sub> (in <sup>4</sup> )	I <sub>zp</sub> (in <sup>4</sup> )	I <sub>w</sub> (in <sup>6</sup> )	S <sub>yp</sub> (in <sup>3</sup> )	S <sub>zp</sub> (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





## Member Design Capacity

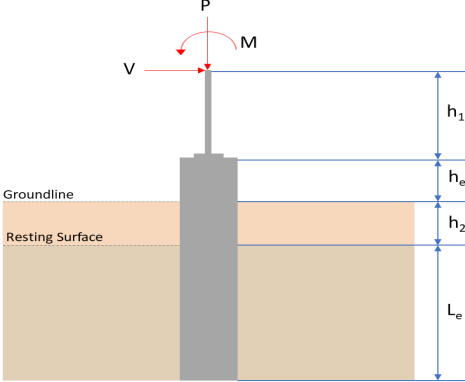
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	105.51	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	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	123.95	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	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	24.97	6.12	40.24	43.62
14	133.20	85.85	24.93	6.12	40.24	43.62
15	133.20	20.65	32.87	6.12	40.24	43.62
16	133.20	20.65	32.87	6.12	40.24	43.62
101	377.97	105.51	83.29	83.29	113.39	113.39
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	123.95	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	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.89	6.12	40.24	43.62
114	133.20	85.85	23.93	6.12	40.24	43.62
115	133.20	69.16	17.22	6.12	40.24	43.62
116	133.20	69.16	17.64	6.12	40.24	43.62
201	377.97	105.51	83.29	83.29	113.39	113.39
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	20.65	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	20.65	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	24.97	6.12	40.24	43.62
214	133.20	85.85	24.93	6.12	40.24	43.62
215	133.20	69.16	19.65	6.12	40.24	43.62
216	133.20	69.16	19.69	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	0.121	0.437	0.032	0.015	0.002	0.502	#13	0.671	Not Required	Pass
2	0.001	0.564	0.080	0.112	0.015	0.645	#13	0.053	Not Required	Pass
3	0.001	0.896	0.013	0.091	0.001	0.910	#13	0.045	Not Required	Pass
4	0.002	0.667	0.025	0.067	0.006	0.670	#13	0.080	Not Required	Pass
5	0.001	0.556	0.028	0.090	0.008	0.567	#13	0.074	Not Required	Pass
6	0.001	0.770	0.007	0.076	0.002	0.775	#13	0.045	Not Required	Pass
7	0.001	0.478	0.019	0.077	0.005	0.480	#13	0.074	Not Required	Pass
8	0.001	0.083	0.029	0.043	0.002	0.112	#13	0.095	Not Required	Pass
9	0.002	0.113	0.025	0.003	0.001	0.140	#13	0.204	Not Required	Pass
10	0.001	0.564	0.034	0.057	0.008	0.595	#13	0.080	Not Required	Pass
11	0.001	0.109	0.026	0.057	0.002	0.135	#13	0.095	Not Required	Pass
12	0.001	0.442	0.070	0.094	0.013	0.513	#13	0.053	Not Required	Pass
13	0.001	0.414	0.062	0.072	0.002	0.467	#13	0.286	Not Required	Pass
14	0.002	0.322	0.062	0.054	0.002	0.356	#13	0.190	Not Required	Pass
15	0.000	0.189	0.037	0.051	0.002	0.221	#13	Not Required	Not Required	Pass
16	0.000	0.138	0.037	0.038	0.002	0.170	#13	Not Required	Not Required	Pass
101	0.111	0.426	0.000	0.014	0.000	0.456	#13	0.671	Not Required	Pass
102	0.000	0.448	0.067	0.094	0.013	0.515	#13	0.053	Not Required	Pass
103	0.001	0.768	0.008	0.077	0.002	0.775	#13	0.045	Not Required	Pass
104	0.001	0.552	0.020	0.056	0.004	0.559	#13	0.080	Not Required	Pass
105	0.001	0.476	0.021	0.077	0.005	0.477	#13	0.074	Not Required	Pass
106	0.001	0.768	0.008	0.077	0.002	0.775	#13	0.045	Not Required	Pass
107	0.001	0.476	0.021	0.077	0.005	0.477	#13	0.074	Not Required	Pass
108	0.001	0.055	0.023	0.034	0.002	0.070	#13	0.095	Not Required	Pass
109	0.002	0.070	0.013	0.001	0.000	0.083	#13	0.204	Not Required	Pass
110	0.001	0.552	0.020	0.056	0.004	0.559	#13	0.080	Not Required	Pass
111	0.001	0.062	0.023	0.048	0.002	0.076	#13	0.095	Not Required	Pass
112	0.000	0.448	0.067	0.094	0.013	0.515	#13	0.053	Not Required	Pass
113	0.001	0.228	0.047	0.062	0.002	0.234	#13	0.286	Not Required	Pass
114	0.002	0.159	0.048	0.045	0.002	0.164	#13	0.286	Not Required	Pass
115	0.001	0.231	0.023	0.048	0.002	0.252	#13	0.473	Not Required	Pass
116	0.001	0.181	0.023	0.034	0.002	0.203	#13	0.473	Not Required	Pass
201	0.121	0.437	0.032	0.015	0.002	0.502	#13	0.671	Not Required	Pass
202	0.001	0.442	0.070	0.094	0.013	0.513	#13	0.053	Not Required	Pass
203	0.001	0.770	0.007	0.076	0.002	0.775	#13	0.045	Not Required	Pass
204	0.001	0.564	0.034	0.057	0.008	0.595	#13	0.080	Not Required	Pass
205	0.001	0.478	0.019	0.077	0.005	0.480	#13	0.074	Not Required	Pass
206	0.001	0.896	0.013	0.091	0.001	0.910	#13	0.045	Not Required	Pass
207	0.001	0.556	0.028	0.090	0.008	0.567	#13	0.074	Not Required	Pass
208	0.000	0.138	0.037	0.038	0.002	0.170	#13	Not Required	Not Required	Pass
209	0.002	0.113	0.025	0.003	0.001	0.140	#13	0.204	Not Required	Pass
210	0.002	0.667	0.025	0.067	0.006	0.670	#13	0.080	Not Required	Pass
211	0.000	0.189	0.037	0.051	0.002	0.221	#13	Not Required	Not Required	Pass
212	0.001	0.564	0.080	0.112	0.015	0.645	#13	0.053	Not Required	Pass
213	0.001	0.414	0.062	0.072	0.002	0.467	#13	0.190	Not Required	Pass
214	0.002	0.322	0.062	0.054	0.002	0.356	#13	0.286	Not Required	Pass
215	0.001	0.210	0.026	0.057	0.002	0.231	#13	0.473	Not Required	Pass
216	0.001	0.162	0.029	0.043	0.002	0.182	#13	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 1285 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>8.448</td> <td>12.782</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.994</td> <td>-1.652</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.163</td> <td>-0.253</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.807</td> <td>-1.252</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.351</td> <td>36.395</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)	8.448	12.782	$V_x$ (kip)	-0.994	-1.652	$V_z$ (kip)	-0.163	-0.253	$M_x$ (kipft)	-0.807	-1.252	$M_z$ (kipft)	21.351	36.395	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.994 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.15828 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.351 \text{ kipft}) + ((-0.994 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3998 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9915 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.163 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.025955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.807 \text{ kipft}) + ((-0.163 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1285 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9366 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.9915 \text{ ft}), (1.9366 \text{ ft})]$$

$$L_{e,req} = 5.992 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.992 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.95872$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.448 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.528 \text{ kip/ft}^2$$

$$q = 0.020 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$Ratio = \frac{(0.528 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.264$$

Status: **PASS**  
Ratio: **0.260**

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:**

$H_o = -0.15828 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.3998 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.3998 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3998 \text{ kipft/ft})) + (4 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2513 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3998 \text{ kipft/ft})) + (3 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3998 \text{ kipft/ft})) + (2 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.26398 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.3998 \text{ kipft/ft})) + ((-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89248 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2513 \text{ ft})}{2}$$

$$p_a = 0.31885 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26398 \text{ kip/ft}^2)}{(0.31885 \text{ kip/ft}^2)}$$

$$Ratio = 0.82793$$

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

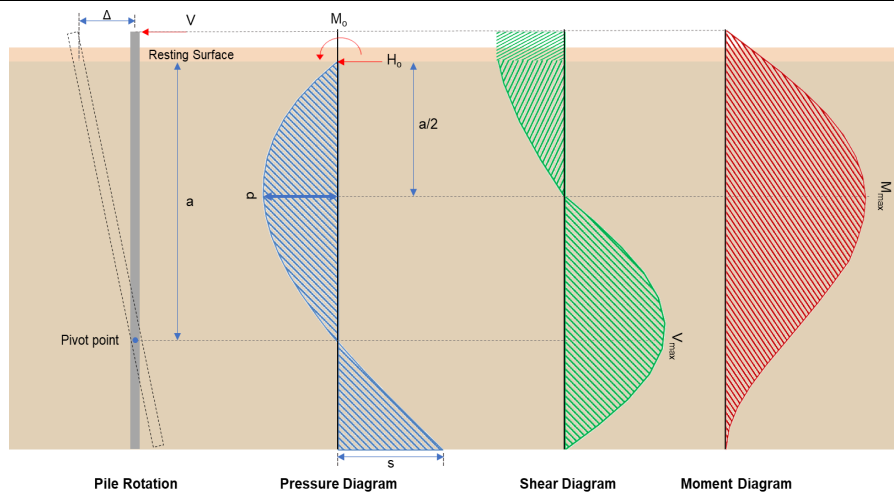
Status: **PASS**  
Ratio: **0.830**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89248 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95198$	<p>Status: <b>PASS</b> Ratio: <b>0.950</b></p>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.025955 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.1285 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.1285 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.1285 \text{ kipft/ft})) + (4 \times (-0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4047 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.1285 \text{ kipft/ft})) + (3 \times (-0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.1285 \text{ kipft/ft})) + (2 \times (-0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.00023516 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.1285 \text{ kipft/ft})) + ((-0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.014559 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4047 \text{ ft})}{2}$ $p_a = 0.33035 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.00023516 \text{ kip/ft}^2)}{(0.33035 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00071186$ <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>0.000</b></p>

$$Ratio = \frac{(0.014559 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.01553$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.652 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.26306 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.395 \text{ kipft}) + ((-1.652 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.7954 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.7954 \text{ kipft/ft})}{(-0.26306 \text{ kip/ft})}$$

$$E = 22.031 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.7954 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.26306 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.7954 \text{ kipft/ft})) + (4 \times (-0.26306 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2495 \text{ ft}$$

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

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

$$V_{max} = ((-0.26306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2495 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2495 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2657 \text{ kip}$$

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

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

$$M_{max} = ((-0.26306 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(22.031 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.103 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.253 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.040287 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.252 \text{ kipft}) + ((-0.253 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19936 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.19936 \text{ kipft/ft})}{(-0.040287 \text{ kip/ft})}$$

$$E = 4.9486 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.19936 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.040287 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19936 \text{ kipft/ft})) + (4 \times (-0.040287 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4047 \text{ ft}$$

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

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

$$V_{max} = ((-0.040287 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4047 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4047 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33246 \text{ kip}$$

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

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

$$M_{max} = ((-0.040287 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(4.9486 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.94852 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(12.782 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.171 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.171 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(12.782 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.004778$$

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

**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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 12.782 \text{ kip} \rightarrow 12782 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12782 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

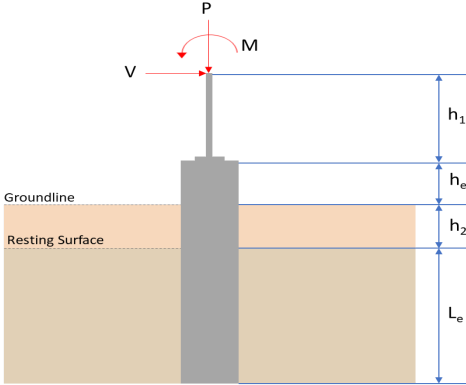
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.2 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.2657 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2657 \text{ kip})}{(111.2 \text{ kip})}$ $\text{Ratio} = 0.065337$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.33246 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33246 \text{ kip})}{(111.2 \text{ kip})}$ $\text{Ratio} = 0.0029897$	<p>Status: <b>PASS</b> Ratio: <b>0.070</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} = 22.103 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(22.103 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.088556$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.94852 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.94852 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0038002$	<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_n</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 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>7.748</td> <td>11.704</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.927</td> <td>-1.561</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>20.843</td> <td>35.444</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_n$ ) (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)	7.748	11.704	$V_x$ (kip)	-0.927	-1.561	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	20.843	35.444	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.927 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.14761 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(20.843 \text{ kipft}) + ((-0.927 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.3189 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation:  <math>L_{e,x} = 5.9688 \text{ ft}</math> - Required depth in x-direction,</p> <p><b>Considering z-direction:</b>  <math>L_{e,z} = 0 \text{ ft}</math> - Required depth in z-direction,</p> <p><b>Minimum embedded depth required:</b>  <math>L_{e,req}</math> - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.9688 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.969 \text{ ft}$ <p><math>L_e</math> - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.25 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.969 \text{ ft})}{(6.25 \text{ ft})}$ $\text{Ratio} = 0.95504$	<p>Status: <b>PASS</b>  Ratio: <b>0.960</b></p>
	<p><b>End-bearing Capacity (ASD)</b>  A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(7.748 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.48425 \text{ kip/ft}^2$ <p><b>Check bearing capacity ratio:</b>  <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.48425 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.24213$	<p>Status: <b>PASS</b>  Ratio: <b>0.240</b></p>
<p>Czerniak</p>	<p><b>Lateral Soil Pressure (ASD):</b>  L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.14761$  kip/ft - Lateral force per length of pile,

$M_o = 3.3189$  kipft/ft - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.3189 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.14761 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3189 \text{ kipft/ft})) + (4 \times (-0.14761 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2481 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3189 \text{ kipft/ft})) + (3 \times (-0.14761 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3189 \text{ kipft/ft})) + (2 \times (-0.14761 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.26136 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.3189 \text{ kipft/ft})) + ((-0.14761 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.87787 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2481 \text{ ft})}{2}$$

$$p_a = 0.31861 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26136 \text{ kip/ft}^2)}{(0.31861 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82031$$

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

$$p_s = R L_e$$

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

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

*Ratio* - Lateral soil capacity

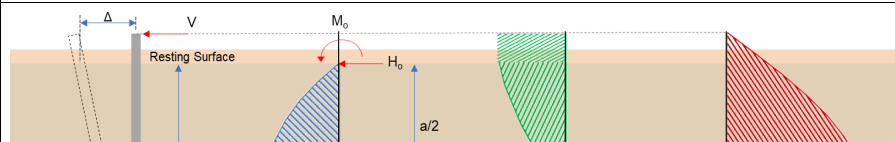
$$\text{Ratio} = \frac{s}{p_s}$$

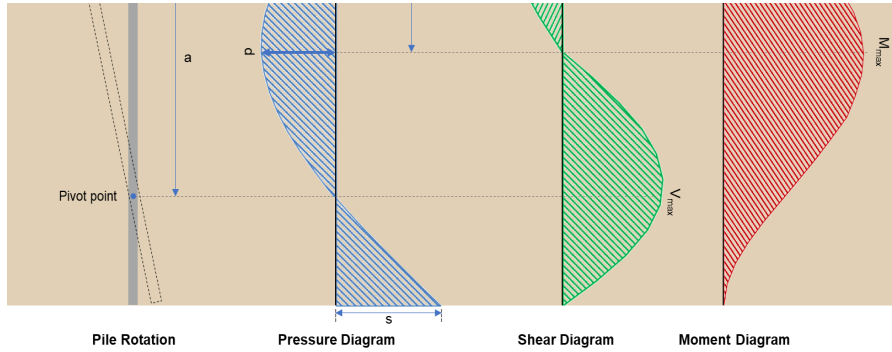
$$\text{Ratio} = \frac{(0.87787 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9364$$

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

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





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.561 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.24857 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(35.444 \text{ kipft}) + ((-1.561 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6439 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.6439 \text{ kipft/ft})}{(-0.24857 \text{ kip/ft})}$$

$$E = 22.706 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.6439 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.24857 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.6439 \text{ kipft/ft})) + (4 \times (-0.24857 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2474 \text{ ft}$$

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

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

$$V_{max} = ((-0.24857 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (22.706 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2474 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (22.706 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2474 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0562 \text{ kip}$$

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

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

$$M_{max} = ((-0.24857 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(22.706 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2474 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (22.706 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2474 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (22.706 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2474 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.482 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(11.704 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.207 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.207 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.704 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.004375$$

Status: **PASS**  
Ratio: **0.000****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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 11.704 \text{ kip} \rightarrow 11704 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11704 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.05 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.05 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.05 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.05 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.11 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.0562 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.0562 \text{ kip})}{(111.11 \text{ kip})}$ $\text{Ratio} = 0.063506$	<p>Status: <b>PASS</b>  Ratio: <b>0.060</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:  <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:**

$M_{max} = 21.482 \text{ kipft}$  - Maximum moment in the x-direction,

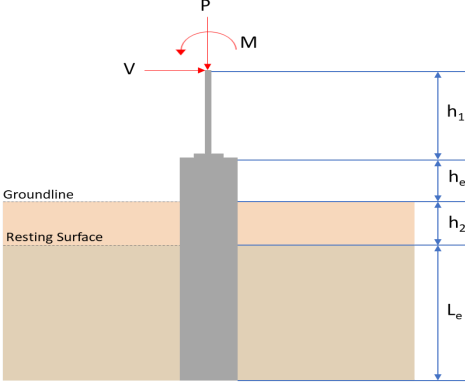
*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(21.482 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.086064$$

Status: **PASS**  
Ratio: **0.090**

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></p> <p>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 1285 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>8.448</td> <td>12.782</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.994</td> <td>-1.652</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.163</td> <td>0.253</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.807</td> <td>1.252</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.351</td> <td>36.396</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><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)	8.448	12.782	$V_x$ (kip)	-0.994	-1.652	$V_z$ (kip)	0.163	0.253	$M_x$ (kipft)	0.807	1.252	$M_z$ (kipft)	21.351	36.396	
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	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><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></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.994 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.15828 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.351 \text{ kipft}) + ((-0.994 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3998 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9915 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.163 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.025955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.807 \text{ kipft}) + ((0.163 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1285 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.4123 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.9915 \text{ ft}), (2.4123 \text{ ft})]$$

$$L_{e,req} = 5.992 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.992 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.95872$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.448 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.528 \text{ kip/ft}^2$$

$$q = 0.020 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$Ratio = \frac{(0.528 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.264$$

Status: **PASS**  
Ratio: **0.260**

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:**

$H_o = -0.15828 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.3998 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.3998 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3998 \text{ kipft/ft})) + (4 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2513 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3998 \text{ kipft/ft})) + (3 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3998 \text{ kipft/ft})) + (2 \times (-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.26398 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.3998 \text{ kipft/ft})) + ((-0.15828 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89248 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2513 \text{ ft})}{2}$$

$$p_a = 0.31885 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26398 \text{ kip/ft}^2)}{(0.31885 \text{ kip/ft}^2)}$$

$$Ratio = 0.82793$$

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

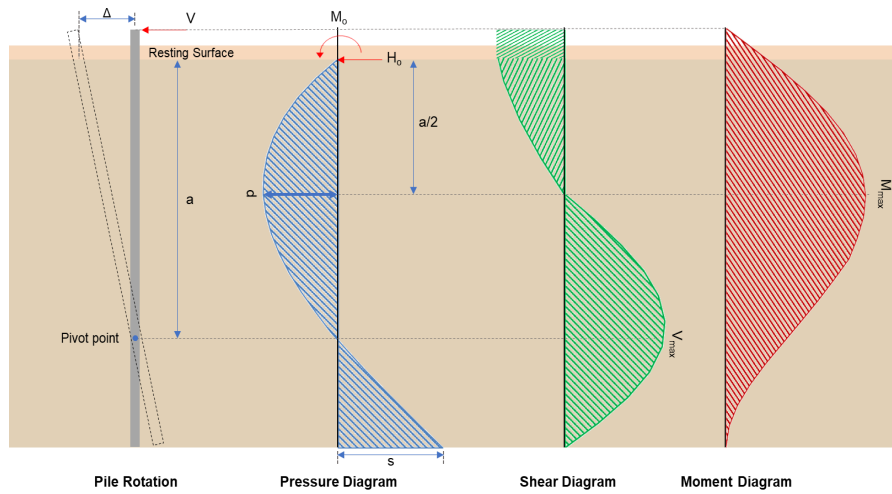
Status: **PASS**  
Ratio: **0.830**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89248 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95198$	<p>Status: <b>PASS</b> Ratio: <b>0.950</b></p>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.025955 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.1285 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.1285 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.025955 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.1285 \text{ kipft/ft})) + (4 \times (0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4047 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.1285 \text{ kipft/ft})) + (3 \times (0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.1285 \text{ kipft/ft})) + (2 \times (0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.027081 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.1285 \text{ kipft/ft})) + ((0.025955 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.064393 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4047 \text{ ft})}{2}$ $p_a = 0.33035 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.027081 \text{ kip/ft}^2)}{(0.33035 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.081975$ <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>0.080</b></p>

$$Ratio = \frac{(0.064393 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.068686$$

Status: **PASS**  
Ratio: **0.070**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.652 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.26306 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.396 \text{ kipft}) + ((-1.652 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.7955 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.7955 \text{ kipft/ft})}{(-0.26306 \text{ kip/ft})}$$

$$E = 22.031 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.7955 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.26306 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.7955 \text{ kipft/ft})) + (4 \times (-0.26306 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2495 \text{ ft}$$

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

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

$$V_{max} = ((-0.26306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2495 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2495 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2659 \text{ kip}$$

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

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

$$M_{max} = ((-0.26306 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(22.031 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (22.031 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.2495 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.104 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.253 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.040287 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.252 \text{ kipft}) + ((0.253 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19936 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.19936 \text{ kipft/ft})}{(0.040287 \text{ kip/ft})}$$

$$E = 4.9486 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.19936 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.040287 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19936 \text{ kipft/ft})) + (4 \times (0.040287 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4047 \text{ ft}$$

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

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

$$V_{max} = ((0.040287 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4047 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4047 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33246 \text{ kip}$$

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

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

$$M_{max} = ((0.040287 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(4.9486 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.9486 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4047 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.94852 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(12.782 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.171 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.171 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(12.782 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.004778$$

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

**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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 12.782 \text{ kip} \rightarrow 12782 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12782 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.2 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.2659 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2659 \text{ kip})}{(111.2 \text{ kip})}$ $\text{Ratio} = 0.065338$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.33246 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33246 \text{ kip})}{(111.2 \text{ kip})}$ $\text{Ratio} = 0.0029897$	<p>Status: <b>PASS</b> Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<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} = 22.104 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(22.104 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.088558$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.94852 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.94852 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0038002$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>