

# Your Project Calculations



Project Name: MTSOLAR\_I16GBFHAL5LB

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=MTSOLAR\\_I16GBFHAL5LB&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/7\\_2023](https://platform.skyciv.com/structural?preload_name=MTSOLAR_I16GBFHAL5LB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=uxKBQ4Rj8AgrfF0uY1nAr6PloB8gCUJTyrIARkrvKLspNufHQVD0L7IbGrn0GCD6](https://platform.skyciv.com/structural-viewer?project_id=uxKBQ4Rj8AgrfF0uY1nAr6PloB8gCUJTyrIARkrvKLspNufHQVD0L7IbGrn0GCD6)

## Array Specification

Product:	Beam
Unique ID:	2P-15-6TOP-SD-24-L-4Hx5W-773H
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	65.00in
Number of Rows:	4
Number of Columns:	5
Total Number of Modules:	20
Desired Tilt Angle:	30
Front Edge Clearance:	6
Total Array Height at Tilt:	12.71 ft
Total Frame Length:	26.50 ft
Frame Weight:	1136 lbs
Array Dimensions N/S:	13.50 ft
Array Dimensions E/W:	27.50 ft
Rail Length:	162.00 in
Rail Spacing:	2.71 ft
Rail Check:	Not Checked

## Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.38 ft
Number of Poles:	2
Pole Spacing:	15 ft

## Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 5.75 ft
Foundation Volume:	6.815 y <sup>3</sup>
Foundation Result:	PASSED
Mount Twist:	0.180052 kip

## Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	10535 Old US Hwy 60, Hawesville, KY 42348, USA
Wind Speed:	100 mph
Snow Load:	15 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.006598 ksf



### Design Disclaimer

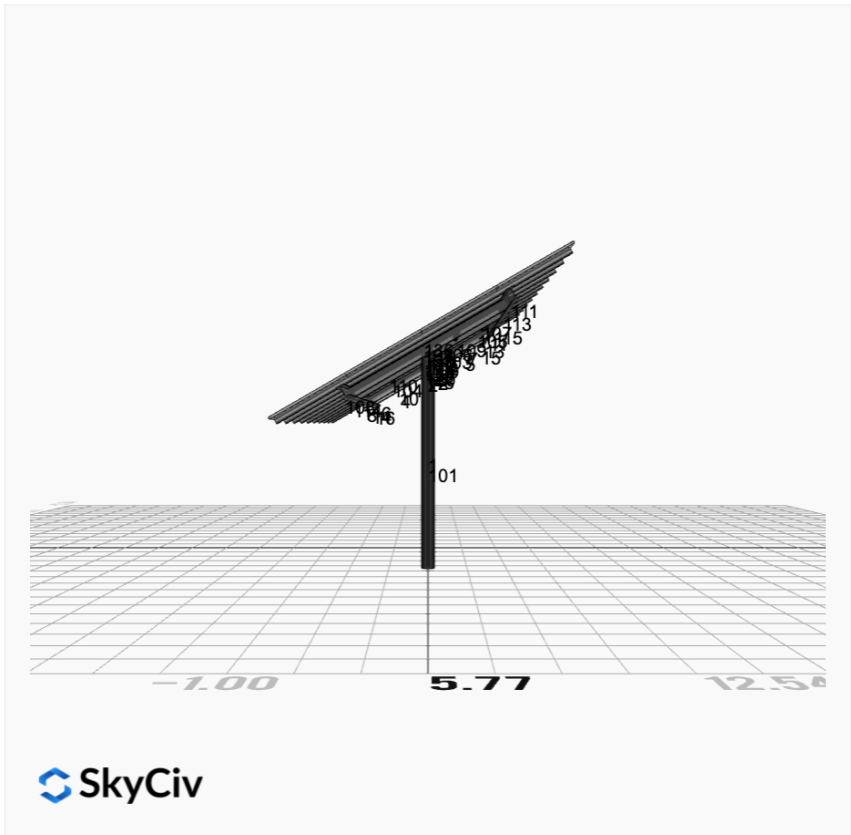
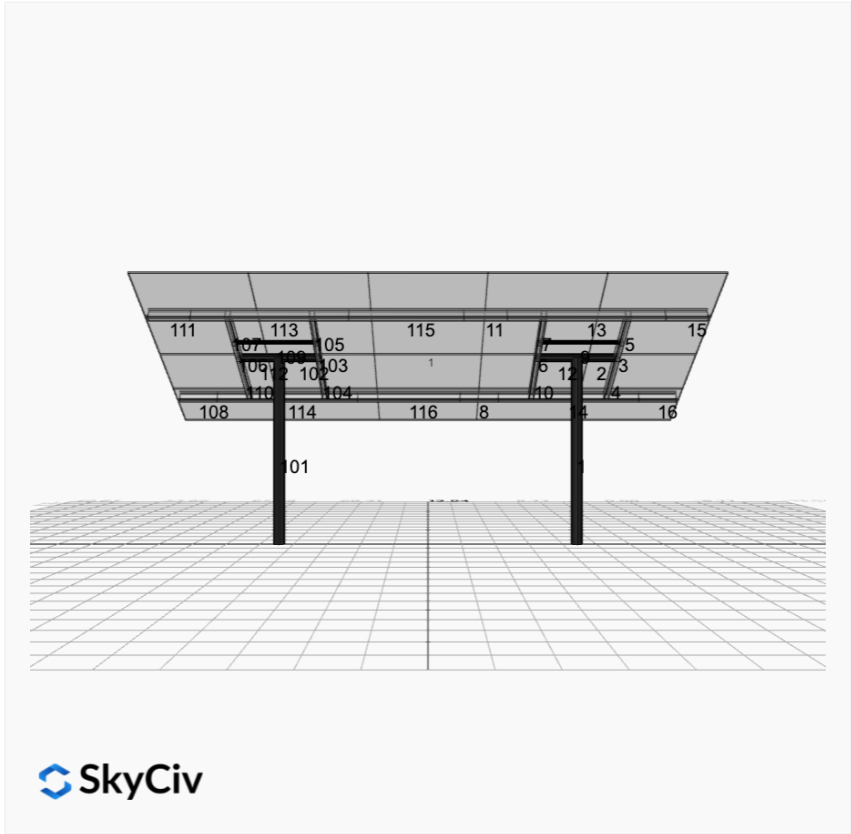
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

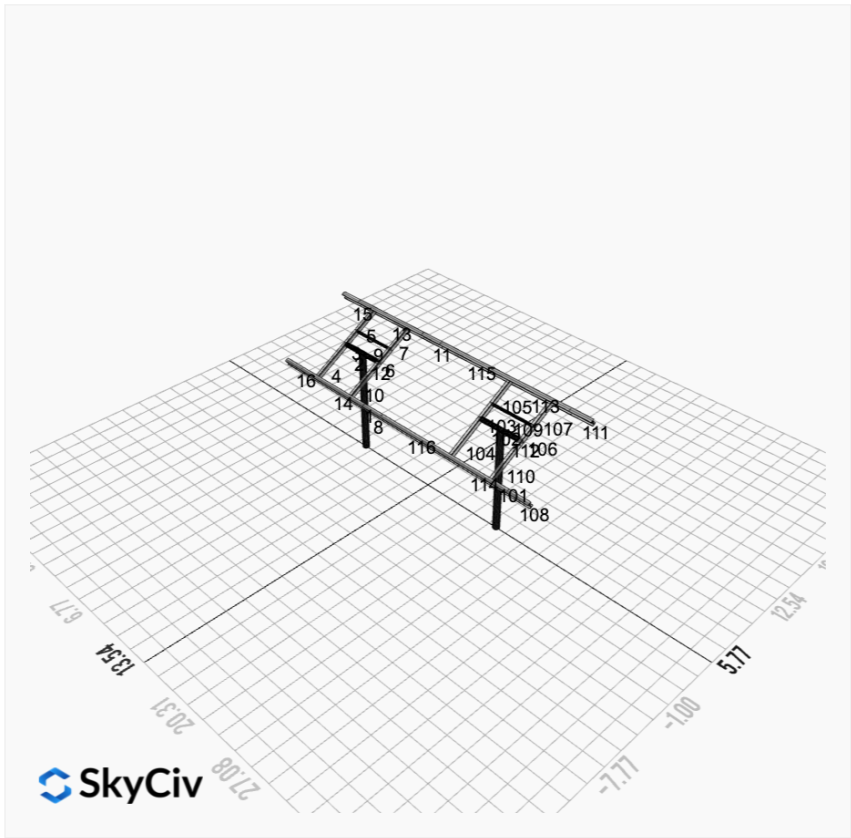
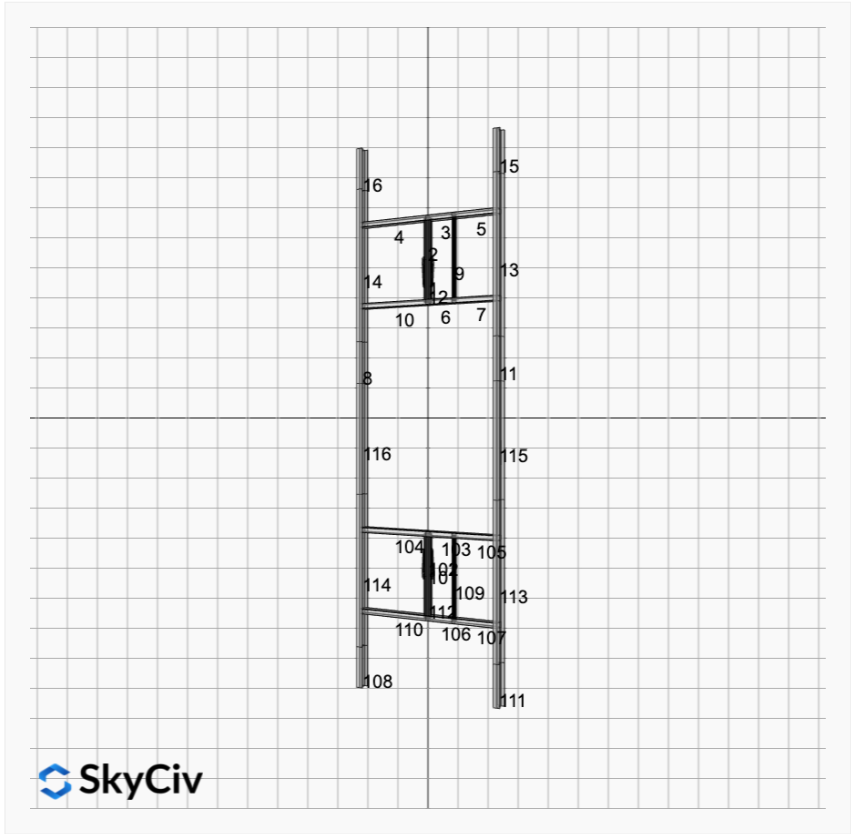
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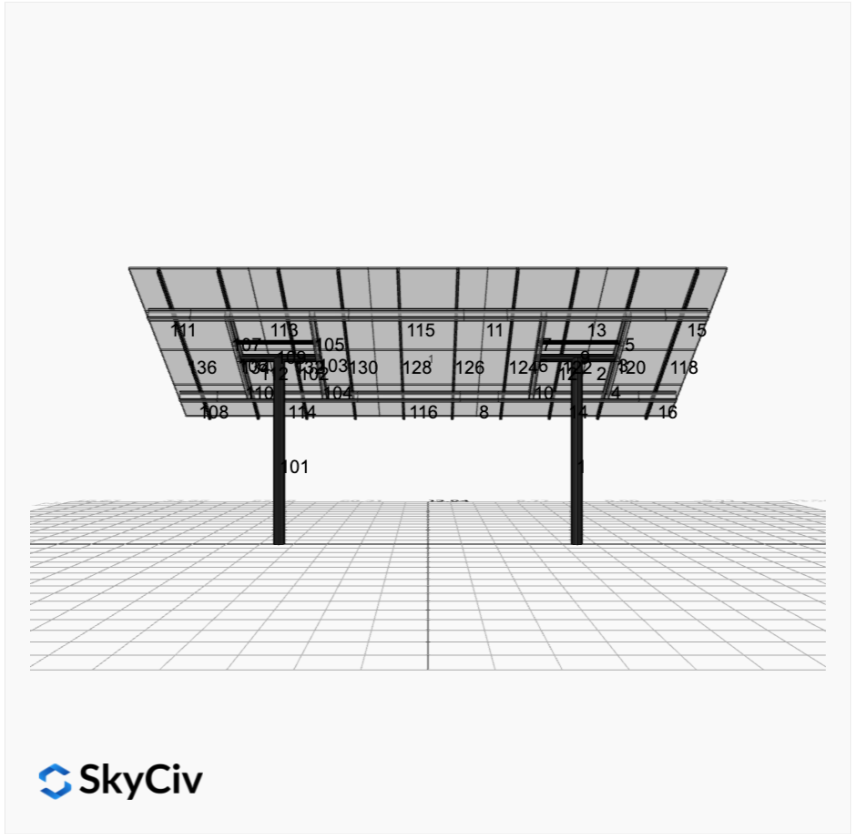
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### Design Notes:

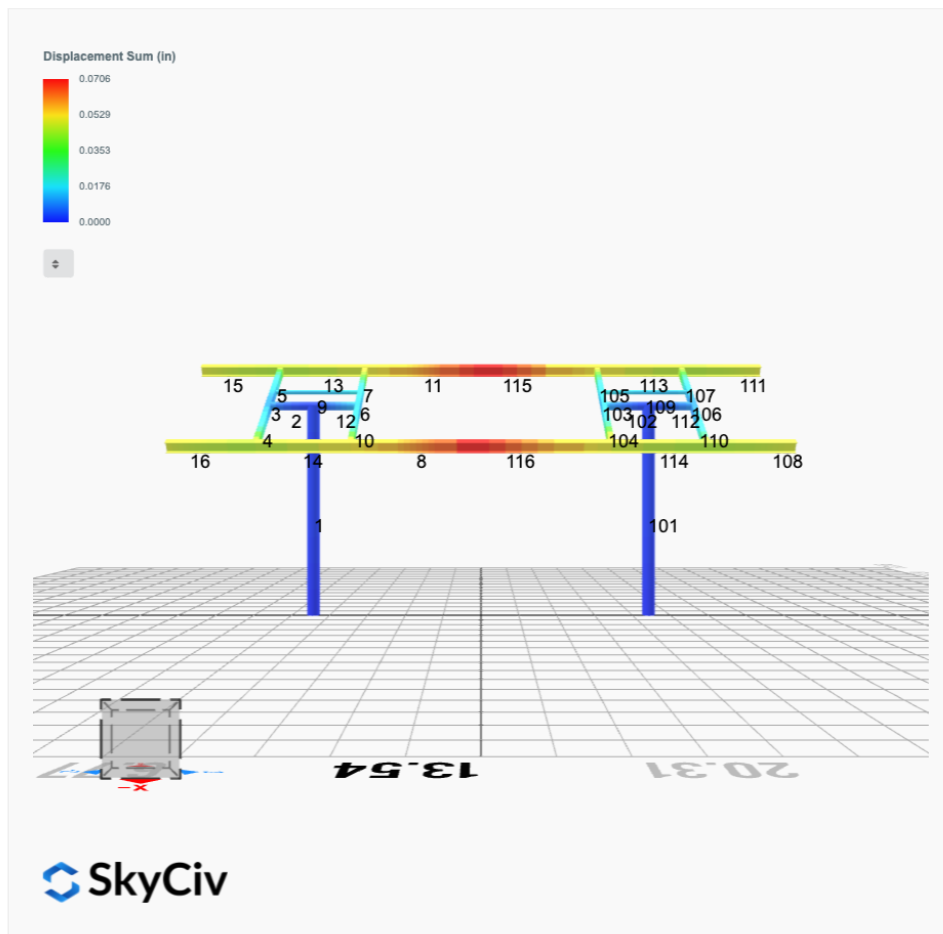
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

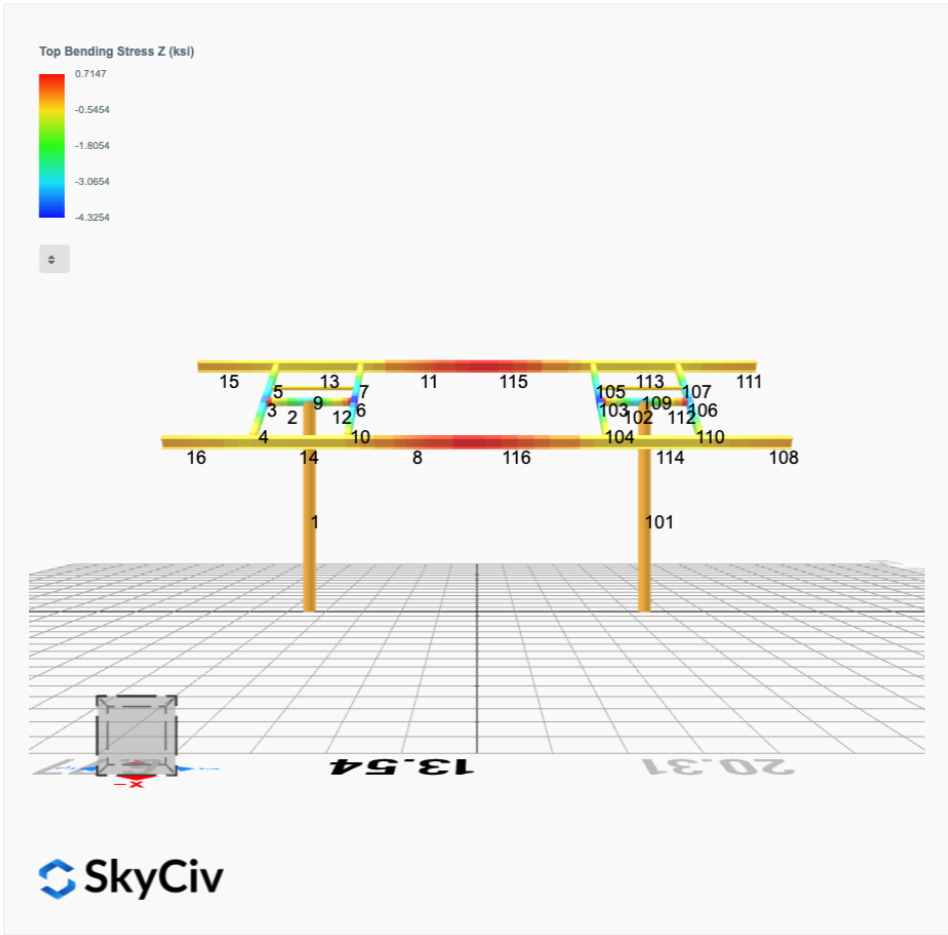


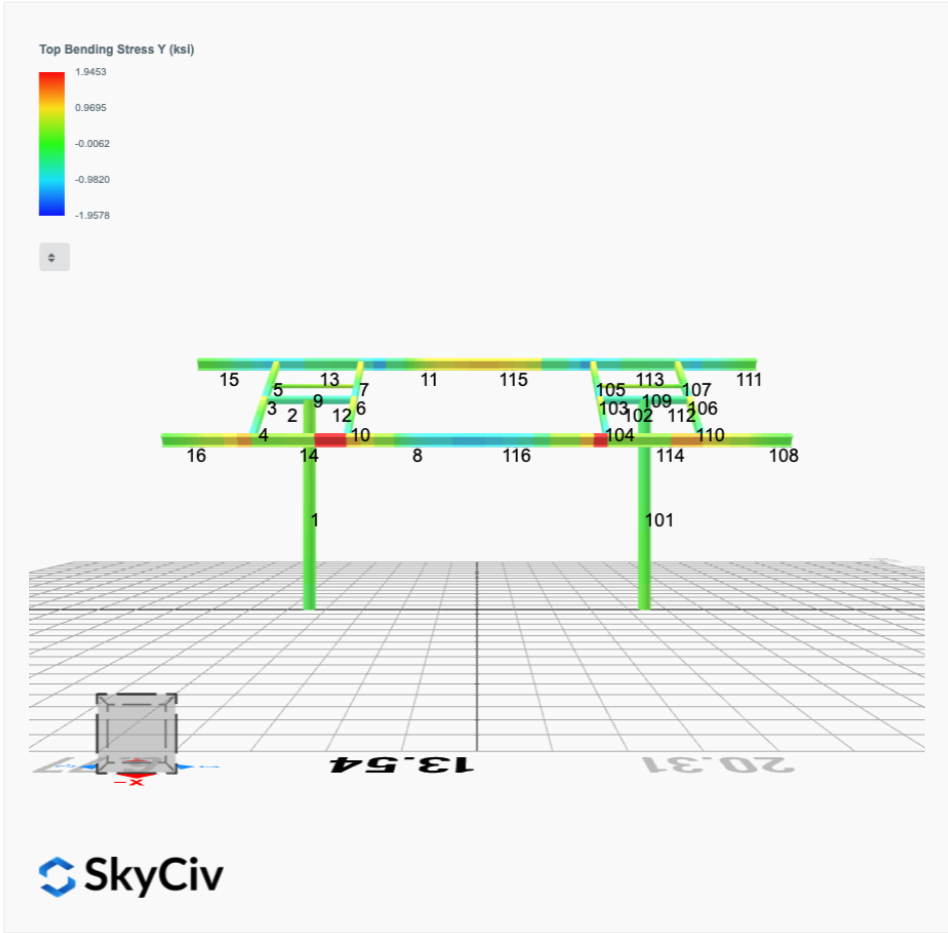


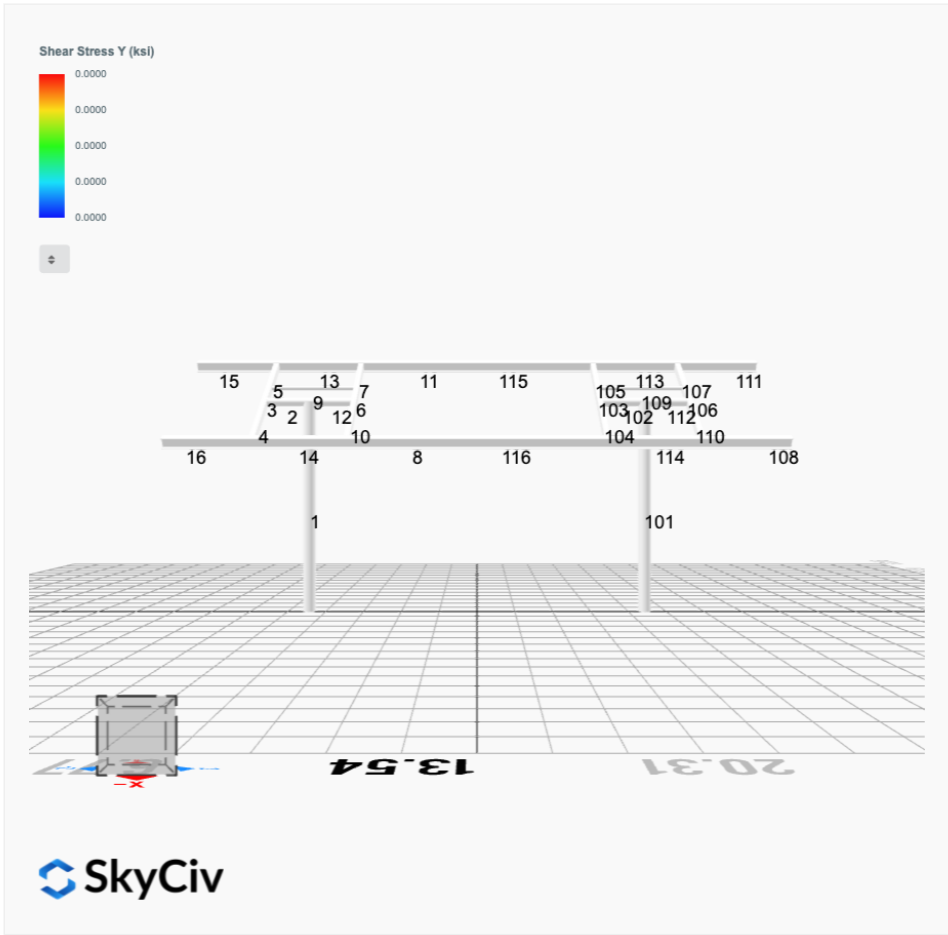


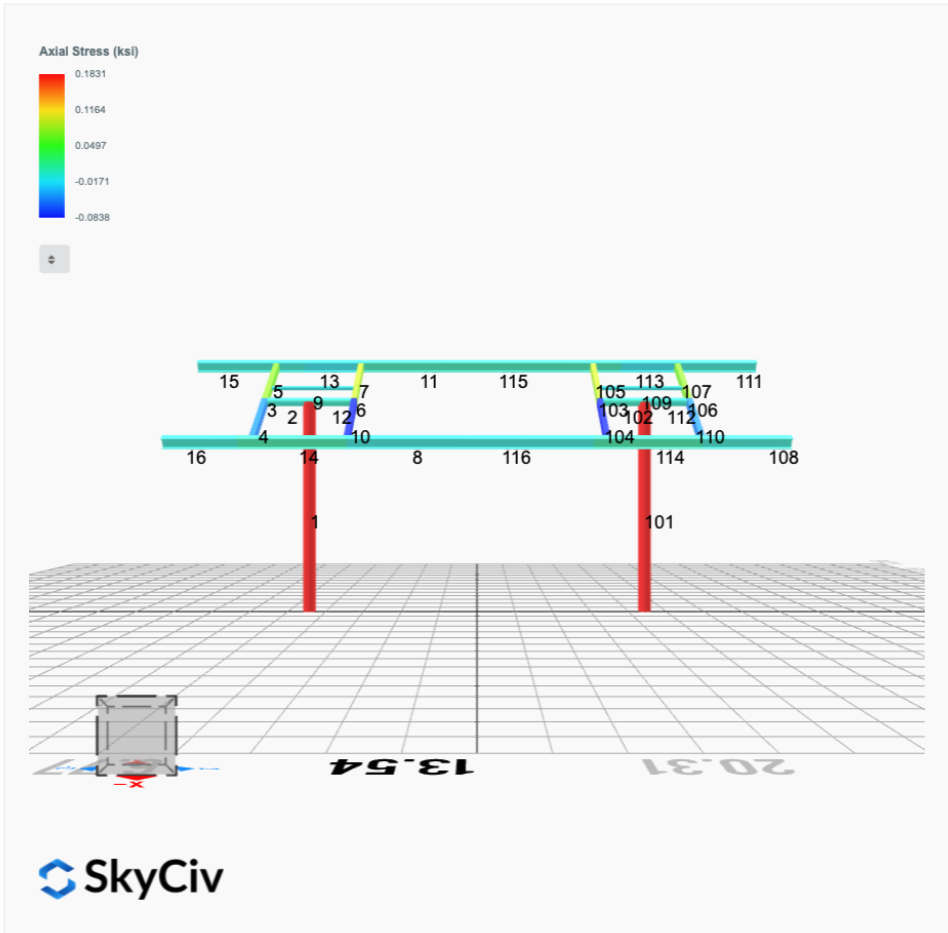
## FEM Results (Envelope Worst Case for each member)











## 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	0.0000	1.4604	0.0209	0.0594	-0.0063	0.0194
ULS: 2. D + L	0.0000	1.4604	0.0209	0.0594	-0.0063	0.0194
ULS: 3. D + (S or Lr or R)	0.0000	2.4825	0.0400	0.1134	-0.0123	0.0211
ULS: 3. D + (S or Lr or R)	0.0000	1.4604	0.0209	0.0594	-0.0063	0.0194
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.2269	0.0352	0.0999	-0.0108	0.0207
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4604	0.0209	0.0594	-0.0063	0.0194
ULS: 5b. D + 0.7E	0.0000	1.4604	0.0209	0.0594	-0.0063	0.0194
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.2269	0.0352	0.0999	-0.0108	0.0207
ULS: 8. 0.6D + 0.7E	0.0000	0.8762	0.0126	0.0356	-0.0038	0.0116
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8026	4.5826	0.0855	0.2363	-0.1082	17.3886
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8026	4.5826	0.0855	0.2363	-0.1082	17.3886
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5451	-1.2158	-0.0341	-0.0908	0.0808	-14.1408
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2876	-0.7698	-0.0254	-0.0672	0.0671	-18.0816
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	4.5686	0.0836	0.2326	-0.0872	13.0476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	4.5686	0.0836	0.2326	-0.0872	13.0476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1588	0.2198	-0.0061	-0.0128	0.0546	-10.5995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9657	0.5543	0.0004	0.0050	0.0443	-13.5551
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	3.8020	0.0694	0.1921	-0.0827	13.0463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	3.8020	0.0694	0.1921	-0.0827	13.0463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1588	-0.5467	-0.0204	-0.0533	0.0591	-10.6007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9657	-0.2122	-0.0138	-0.0355	0.0487	-13.5564
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8026	3.9984	0.0771	0.2125	-0.1056	17.3809
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8026	3.9984	0.0771	0.2125	-0.1056	17.3809
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5451	-1.7999	-0.0425	-0.1146	0.0834	-14.1486
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2876	-1.3539	-0.0338	-0.0909	0.0696	-18.0894

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	7.4672
Shear X	-3.0044
Shear Z	0.1423
Moment X	0.3938
Moment Y (Twist)	0.1802
Moment Z	30.5667

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.5826
Shear X	-1.8026
Shear Z	0.0855
Moment X	0.2363
Moment Y (Twist)	0.1082
Moment Z	18.0894

## Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.4604	-0.0209	-0.0594	0.0063	0.0194
ULS: 2. D + L	-0.0000	1.4604	-0.0209	-0.0594	0.0063	0.0194
ULS: 3. D + (S or Lr or R)	-0.0000	2.4825	-0.0400	-0.1134	0.0123	0.0211
ULS: 3. D + (S or Lr or R)	-0.0000	1.4604	-0.0209	-0.0594	0.0063	0.0194
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.2269	-0.0352	-0.0999	0.0108	0.0207
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.4604	-0.0209	-0.0594	0.0063	0.0194
ULS: 5b. D + 0.7E	-0.0000	1.4604	-0.0209	-0.0594	0.0063	0.0194

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.2269	-0.0352	-0.0999	0.0108	0.0207
ULS: 8. 0.6D + 0.7E	-0.0000	0.8762	-0.0126	-0.0356	0.0038	0.0116
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8026	4.5826	-0.0855	-0.2363	0.1082	17.3886
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8026	4.5826	-0.0855	-0.2363	0.1082	17.3886
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5451	-1.2158	0.0341	0.0908	-0.0808	-14.1408
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2876	-0.7698	0.0254	0.0672	-0.0671	-18.0816
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	4.5686	-0.0836	-0.2326	0.0872	13.0476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	4.5686	-0.0836	-0.2326	0.0872	13.0476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1588	0.2198	0.0061	0.0128	-0.0546	-10.5995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9657	0.5543	-0.0004	-0.0050	-0.0443	-13.5551
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	3.8020	-0.0694	-0.1921	0.0827	13.0463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	3.8020	-0.0694	-0.1921	0.0827	13.0463
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1588	-0.5467	0.0204	0.0533	-0.0591	-10.6007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9657	-0.2122	0.0138	0.0355	-0.0487	-13.5564
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8026	3.9984	-0.0771	-0.2125	0.1056	17.3809
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8026	3.9984	-0.0771	-0.2125	0.1056	17.3809
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5451	-1.7999	0.0425	0.1146	-0.0834	-14.1486
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2876	-1.3539	0.0338	0.0909	-0.0696	-18.0894

#### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	7.4672
Shear X	-3.0043
Shear Z	-0.1423
Moment X	-0.3937
Moment Y (Twist)	0.1801
Moment Z	30.5674

#### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.5826
Shear X	-1.8026
Shear Z	-0.0855
Moment X	-0.2363
Moment Y (Twist)	0.1082
Moment Z	18.0894

## Project Details

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

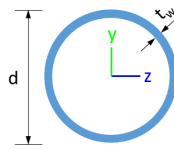


## Design Input Information

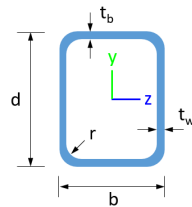
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$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
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Design Materials			
ID	E (ksi)	$F_y$ (ksi)	$F_u$ (ksi)
1	29000	50	65

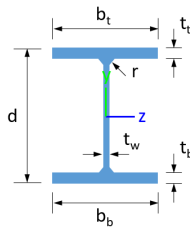
### Section Dimensions



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



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



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

### Section Properties

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

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

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	7	19.69	19.69	9.38	-	3	2	1
2	4	4.20	4.20	2.00	-	3	2	1
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.42,1.16,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18	3	2	1
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.67,1.66,1.77,1.67,1.67,1.66,1.55,1.67,1.67,2.38,1.68,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.63	3	2	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.66,1.67,1.67,2.40,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	3	2	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.38,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	3	2	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,2.09,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	3	2	1
8	18	1.33	1.33	2.05	1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,2.34,1.28,1.28,1.28,1.16,1.28,1.28,1.32,1.30,1.28,1.28,1.27,1.36,1.28,1.28,1.28,1.21	3	2	1
9	1	2.60	2.60	4.00	-	3	2	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.83,1.67,1.67,1.66,1.59,1.67,1.67,2.16,1.68,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.64	3	2	1
11	18	1.33	1.33	2.05	1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.29,1.28,1.28,1.28,1.29,1.28,1.28,1.30,1.29,1.28,1.28,1.28,1.29,1.28,1.28,1.29	3	2	1
12	4	1.30	1.30	2.00	-	3	2	1
13	18	4.88	4.00	7.50	1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.24,1.28,1.23,1.23,1.24,1.27,1.23,1.23,1.20,1.47,1.23,1.23,1.24,1.29,1.23,1.23,1.24,1.27	3	2	1
14	18	4.88	4.00	7.50	1.22,1.22,1.22,1.22,1.22,1.22,1.23,1.23,1.23,1.20,1.23,1.23,1.23,1.08,1.22,1.22,1.17,1.28,1.23,1.23,1.24,2.03,1.23,1.23,1.23,1.05	3	2	1
15	18	4.20	4.20	2.00	2.33,2.33	3	2	1
16	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	3	2	1
101	7	19.69	19.69	9.38	-	3	2	1
102	4	1.30	1.30	2.00	-	3	2	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.38,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	3	2	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.83,1.67,1.67,1.66,1.59,1.67,1.67,2.16,1.68,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.64	3	2	1
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,2.09,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	3	2	1
106	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.42,1.16,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18	3	2	1
107	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,2.40,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	3	2	1



115	120.00	102.67	21.57	6.45	30.09	45.74
116	120.60	102.67	21.57	6.45	30.09	45.74

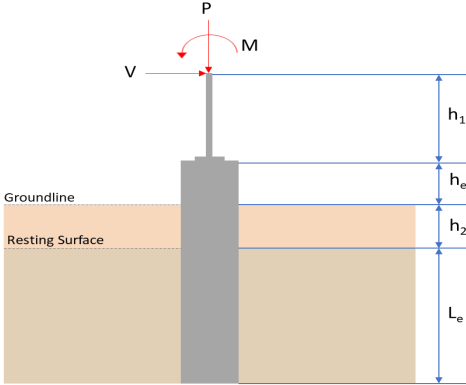
## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.067	0.723	0.022	0.040	0.002	0.737	#13	0.526	Not Required	Pass
2	0.000	0.359	0.173	0.080	0.035	0.532	#13	0.167	Not Required	Pass
3	0.005	0.629	0.054	0.063	0.009	0.657	#13	0.044	Not Required	Pass
4	0.004	0.627	0.064	0.063	0.009	0.681	#13	0.078	Not Required	Pass
5	0.005	0.390	0.049	0.063	0.006	0.394	#13	0.073	Not Required	Pass
6	0.006	0.688	0.086	0.070	0.015	0.750	#13	0.044	Not Required	Pass
7	0.006	0.426	0.085	0.069	0.012	0.441	#13	0.073	Not Required	Pass
8	0.001	0.106	0.038	0.036	0.005	0.130	#13	0.088	Not Required	Pass
9	0.001	0.059	0.043	0.001	0.001	0.102	#13	0.132	Not Required	Pass
10	0.006	0.686	0.083	0.069	0.012	0.711	#13	0.078	Not Required	Pass
11	0.001	0.105	0.038	0.036	0.005	0.128	#13	0.088	Not Required	Pass
12	0.001	0.417	0.184	0.089	0.036	0.601	#13	0.052	Not Required	Pass
13	0.001	0.111	0.083	0.053	0.007	0.134	#21	0.265	Not Required	Pass
14	0.002	0.115	0.082	0.053	0.007	0.132	#21	0.177	Not Required	Pass
15	0.000	0.025	0.018	0.019	0.002	0.036	#21	Not Required	Not Required	Pass
16	0.000	0.025	0.018	0.019	0.002	0.036	#21	Not Required	Not Required	Pass
101	0.067	0.723	0.022	0.040	0.002	0.737	#13	0.526	Not Required	Pass
102	0.001	0.417	0.184	0.089	0.036	0.601	#13	0.052	Not Required	Pass
103	0.006	0.688	0.086	0.070	0.015	0.750	#13	0.044	Not Required	Pass
104	0.006	0.686	0.083	0.069	0.012	0.711	#13	0.078	Not Required	Pass
105	0.006	0.426	0.085	0.069	0.012	0.441	#13	0.073	Not Required	Pass
106	0.005	0.629	0.054	0.063	0.009	0.657	#13	0.044	Not Required	Pass
107	0.005	0.390	0.049	0.063	0.006	0.394	#13	0.073	Not Required	Pass
108	0.000	0.025	0.018	0.019	0.002	0.036	#21	Not Required	Not Required	Pass
109	0.001	0.059	0.043	0.001	0.001	0.102	#13	0.132	Not Required	Pass
110	0.004	0.627	0.064	0.063	0.009	0.681	#13	0.078	Not Required	Pass
111	0.000	0.025	0.018	0.019	0.002	0.036	#21	Not Required	Not Required	Pass
112	0.000	0.359	0.173	0.080	0.035	0.532	#13	0.167	Not Required	Pass
113	0.001	0.111	0.083	0.053	0.007	0.134	#21	0.177	Not Required	Pass
114	0.002	0.115	0.082	0.053	0.007	0.132	#21	0.265	Not Required	Pass
115	0.001	0.129	0.050	0.036	0.005	0.159	#13	0.235	Not Required	Pass
116	0.001	0.130	0.050	0.036	0.005	0.162	#13	0.235	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 <sub>y</sub>	Specified minimum yield stress
F <sub>u</sub>	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I <sub>yp</sub>	Moment of inertia about the Y axes
I <sub>zp</sub>	Moment of inertia about the Z axes
I <sub>w</sub>	Warping constant
S <sub>yp</sub>	Plastic section modulus about the Y axis
S <sub>zp</sub>	Plastic section modulus about the Z axis
KL	Effective length
C <sub>b</sub>	Buckling modification factor (from all load combinations)
L <sub>r</sub>	Length between braced points

$l$	Length between brace 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 = 5.75</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>4.583</td> <td>7.467</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.803</td> <td>-3.004</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.086</td> <td>0.142</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.236</td> <td>0.394</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>18.089</td> <td>30.567</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.583	7.467	$V_x$ (kip)	-1.803	-3.004	$V_z$ (kip)	0.086	0.142	$M_x$ (kipft)	0.236	0.394	$M_z$ (kipft)	18.089	30.567	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.803 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.2871 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(18.089 \text{ kipft}) + ((-1.803 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.8804 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.2025 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.086 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.013694 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.236 \text{ kipft}) + ((0.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.03758 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.632 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.2025 \text{ ft}), (1.632 \text{ ft})]$$

$$L_{e,req} = 5.203 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.203 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.90487$$

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

**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

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

$$q = \frac{(4.583 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28644 \text{ kip/ft}^2$$

$$q = 0.20944 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28644 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14322$$

Status: **PASS**  
Ratio: **0.140**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.2871 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.8804 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.8804 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.8804 \text{ kipft/ft})) + (4 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9658 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.8804 \text{ kipft/ft})) + (3 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.8804 \text{ kipft/ft})) + (2 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18333 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.8804 \text{ kipft/ft})) + ((-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.74586 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9658 \text{ ft})}{2}$$

$$p_a = 0.29744 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18333 \text{ kip/ft}^2)}{(0.29744 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61638$$

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

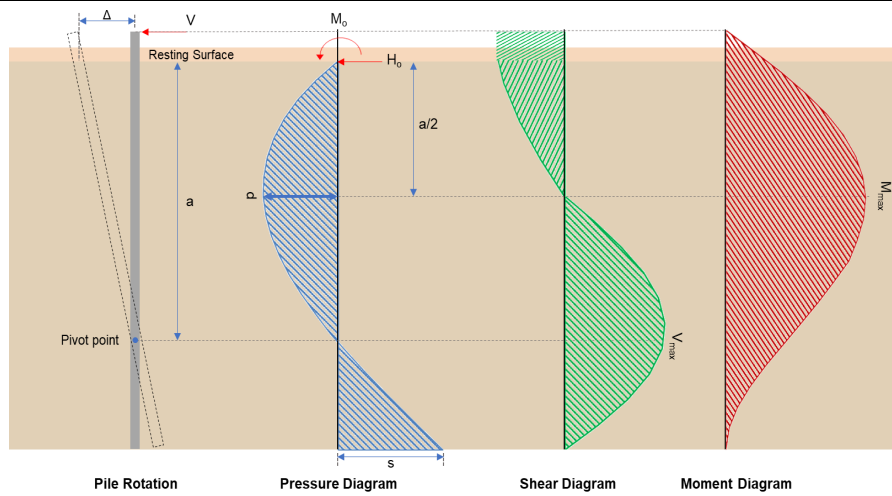
Status: **PASS**  
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74586 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.86476$	Status: <b>PASS</b> Ratio: <b>0.860</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.013694 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.03758 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.03758 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.013694 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.03758 \text{ kipft/ft})) + (4 \times (0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1126 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.03758 \text{ kipft/ft})) + (3 \times (0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.03758 \text{ kipft/ft})) + (2 \times (0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.012543 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.03758 \text{ kipft/ft})) + ((0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.027929 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1126 \text{ ft})}{2}$ $p_a = 0.30844 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.012543 \text{ kip/ft}^2)}{(0.30844 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.040665$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.040</b>

$$Ratio = \frac{(0.027929 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.032382$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.004 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47834 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(30.567 \text{ kipft}) + ((-3.004 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.8674 \text{ kipft/ft}$$

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

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

$$E = \frac{(4.8674 \text{ kipft/ft})}{(-0.47834 \text{ kip/ft})}$$

$$E = 10.175 \text{ ft}$$

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

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

$$a = \frac{(4 \times (4.8674 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.47834 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.8674 \text{ kipft/ft})) + (4 \times (-0.47834 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9645 \text{ ft}$$

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

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

$$V_{max} = ((-0.47834 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9645 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9645 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2537 \text{ kip}$$

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

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

$$M_{max} = ((-0.47834 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(10.175 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 19.855 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.022611 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.394 \text{ kipft}) + ((0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.062739 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.062739 \text{ kipft/ft})}{(0.022611 \text{ kip/ft})}$$

$$E = 2.7746 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.062739 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.022611 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.062739 \text{ kipft/ft})) + (4 \times (0.022611 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1113 \text{ ft}$$

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

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

$$V_{max} = ((0.022611 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1113 \text{ ft})}{(5.75 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1113 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.13752 \text{ kip}$$

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

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

$$M_{max} = ((0.022611 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.7746 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.34901 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.467 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.348 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.348 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(7.467 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0027912$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.467 \text{ kip} \rightarrow 7467 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7467 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.48 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

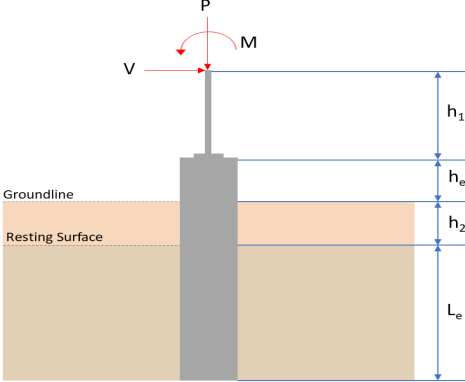
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.48 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.48 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.74 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.2537 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2537 \text{ kip})}{(110.74 \text{ kip})}$ $\text{Ratio} = 0.0655$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.13752 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.13752 \text{ kip})}{(110.74 \text{ kip})}$ $\text{Ratio} = 0.0012418$	<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} = 19.855 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.855 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.079548$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.34901 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.34901 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0013983$	<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 = 5.75</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>4.583</td> <td>7.467</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.803</td> <td>-3.004</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.086</td> <td>-0.142</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.236</td> <td>-0.394</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>18.089</td> <td>30.567</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.583	7.467	$V_x$ (kip)	-1.803	-3.004	$V_z$ (kip)	-0.086	-0.142	$M_x$ (kipft)	-0.236	-0.394	$M_z$ (kipft)	18.089	30.567	
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$M_x$ (kipft)	-0.236	-0.394																										
$M_z$ (kipft)	18.089	30.567																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.803 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.2871 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(18.089 \text{ kipft}) + ((-1.803 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.8804 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.2025 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.086 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.013694 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.236 \text{ kipft}) + ((-0.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.03758 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2548 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.2025 \text{ ft}), (1.2548 \text{ ft})]$$

$$L_{e,req} = 5.203 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.203 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.90487$$

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

**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

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

$$q = \frac{(4.583 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28644 \text{ kip/ft}^2$$

$$q = 0.20944 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28644 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14322$$

Status: **PASS**  
Ratio: **0.140**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.2871 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.8804 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.8804 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.8804 \text{ kipft/ft})) + (4 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9658 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.8804 \text{ kipft/ft})) + (3 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.8804 \text{ kipft/ft})) + (2 \times (-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18333 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.8804 \text{ kipft/ft})) + ((-0.2871 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.74586 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9658 \text{ ft})}{2}$$

$$p_a = 0.29744 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18333 \text{ kip/ft}^2)}{(0.29744 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61638$$

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

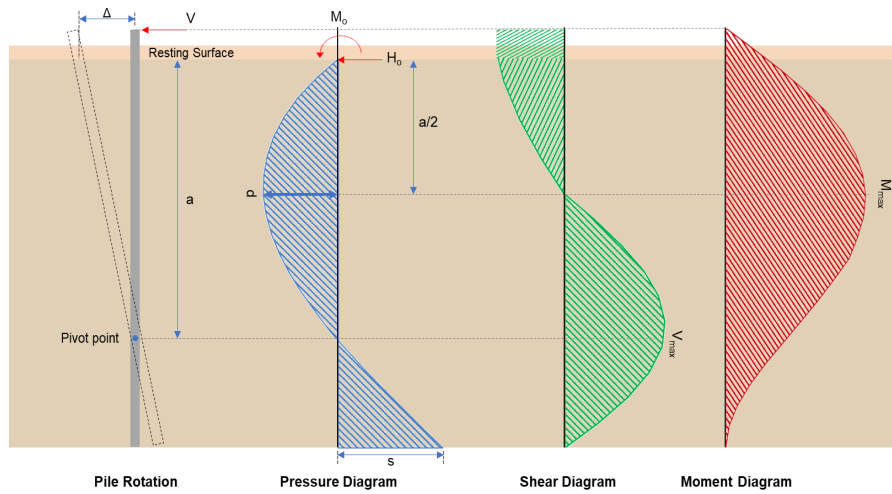
Status: **PASS**  
Ratio: **0.620**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74586 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.86476$	Status: <b>PASS</b> Ratio: <b>0.860</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.013694 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.03758 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.03758 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.013694 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.03758 \text{ kipft/ft})) + (4 \times (-0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1126 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.03758 \text{ kipft/ft})) + (3 \times (-0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.03758 \text{ kipft/ft})) + (2 \times (-0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = -0.0037415 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.03758 \text{ kipft/ft})) + ((-0.013694 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = -0.00065019 \text{ kip/ft}^2$ <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> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1126 \text{ ft})}{2}$ $p_a = 0.30844 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0037415 \text{ kip/ft}^2)}{(0.30844 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.01213$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>-0.010</b>

$$Ratio = \frac{(-0.00065019 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.00075384$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.004 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47834 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(30.567 \text{ kipft}) + ((-3.004 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.8674 \text{ kipft/ft}$$

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

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

$$E = \frac{(4.8674 \text{ kipft/ft})}{(-0.47834 \text{ kip/ft})}$$

$$E = 10.175 \text{ ft}$$

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

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

$$a = \frac{(4 \times (4.8674 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.47834 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.8674 \text{ kipft/ft})) + (4 \times (-0.47834 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9645 \text{ ft}$$

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

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

$$V_{max} = ((-0.47834 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9645 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9645 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.2537 \text{ kip}$$

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

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

$$M_{max} = ((-0.47834 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(10.175 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.175 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9645 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.855 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.022611 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.394 \text{ kipft}) + ((-0.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.062739 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.062739 \text{ kipft/ft})}{(-0.022611 \text{ kip/ft})}$$

$$E = 2.7746 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.062739 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.022611 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.062739 \text{ kipft/ft})) + (4 \times (-0.022611 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1113 \text{ ft}$$

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

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

$$V_{max} = ((-0.022611 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1113 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1113 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.13752 \text{ kip}$$

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

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

$$M_{max} = ((-0.022611 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.7746 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.7746 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1113 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.34901 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.467 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.348 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.348 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$\text{Ratio} = \frac{(7.467 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0027912$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.467 \text{ kip} \rightarrow 7467 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7467 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.48 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.48 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{ytik} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.48 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.74 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.2537 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.2537 \text{ kip})}{(110.74 \text{ kip})}$ $\text{Ratio} = 0.0655$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.13752 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.13752 \text{ kip})}{(110.74 \text{ kip})}$ $\text{Ratio} = 0.0012418$	<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} = 19.855 \text{kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(19.855 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.079548$	<p>Status: <b>PASS</b>          Ratio: <b>0.080</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.34901 \text{kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.34901 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0013983$	<p>Status: <b>PASS</b>          Ratio: <b>0.000</b></p>