

# Project Details



**Project Name:** Tim Tyler

**Date:** Wed Oct 23 2024

**Location:** 48170 144th St, Big Stone City, SD 57216, USA

**Number of Modules:** 24

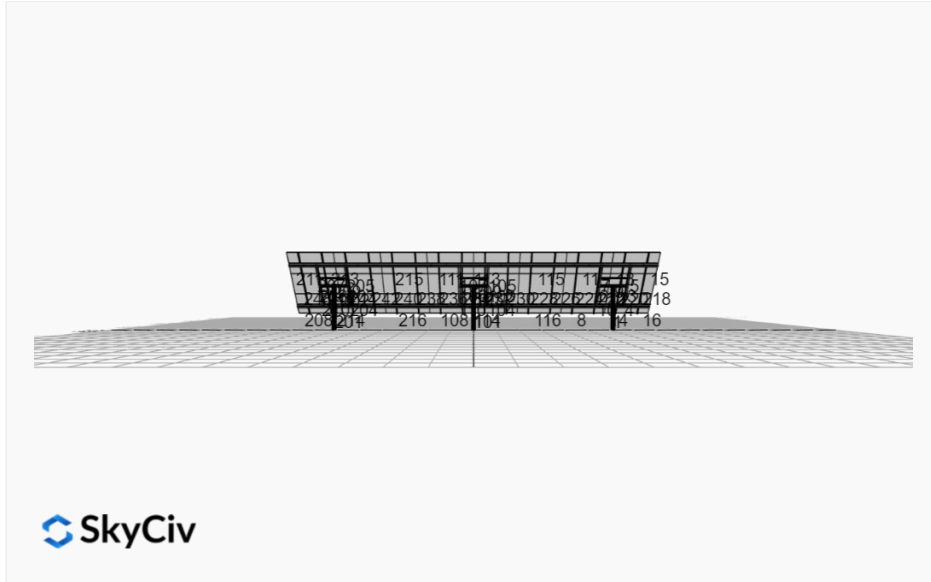
**Number of Poles:** 3

**Unique ID:** 3P-19.75-6TOP-SD-24-L-3Hx8W-GC10

**Date Sold:**

**Dealer:** \_\_\_\_\_

\_\_\_\_\_



<b>Array Dimensions N/S</b>	11.13 ft
<b>Array Dimensions E/W</b>	51.33 ft
<b>Winter Tilt Angle</b>	50
<b>Front Edge Clearance</b>	2 ft

## MT Solar Bill of Materials (3P-19.75-6TOP-SD-24-L-3Hx8W-GC10)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	3
MTS-HF-SD	H-Frame Assembly-SD	3
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	4
MTS-SD-Splice-57	57IN SD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	8

## Rail Bill of Materials

Part	Qty
Rails (132in)	16
Rail Attachment	32
Module Mid Clamp	32
Module End Clamp	32
Ground Lug	8

## Site Details:



**Site Address:** 48170 144th St, Big Stone City, SD 57216, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	44.00 in
<b>Module Length:</b>	76.00in
<b>Number of Rows:</b>	3
<b>Number of Columns:</b>	8
<b>Total Number of Modules:</b>	24
<b>Winter Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	2
<b>Total Array Height at Tilt:</b>	10.52 ft
<b>Total Frame Length:</b>	51.00 ft
<b>Frame Weight:</b>	2233 lbs
<b>Array Dimensions N/S:</b>	11.13 ft
<b>Array Dimensions E/W:</b>	51.33 ft
<b>Rail Length:</b>	133.50 in
<b>Rail Spacing:</b>	3.21 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	6.26 ft
<b>Number of Poles:</b>	3
<b>Pole Spacing:</b>	19.75 ft

### Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.25 ft Pile 2: 6.75 ft Pile 3: 6.25 ft
<b>Foundation Volume:</b>	5.040 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	48170 144th St, Big Stone City, SD 57216, USA
<b>Wind Speed:</b>	105 mph
<b>Snow Load:</b>	50 psf

### **Design Disclaimer**

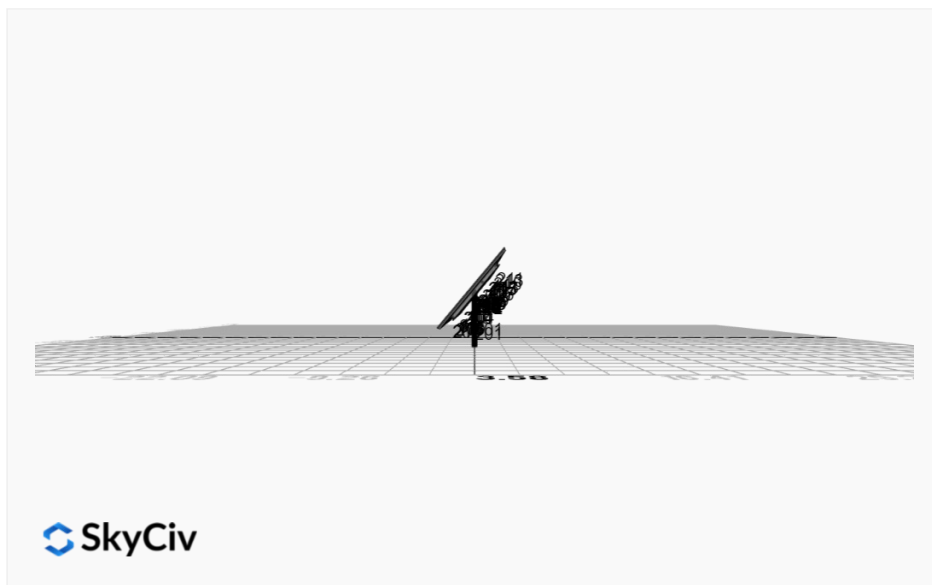
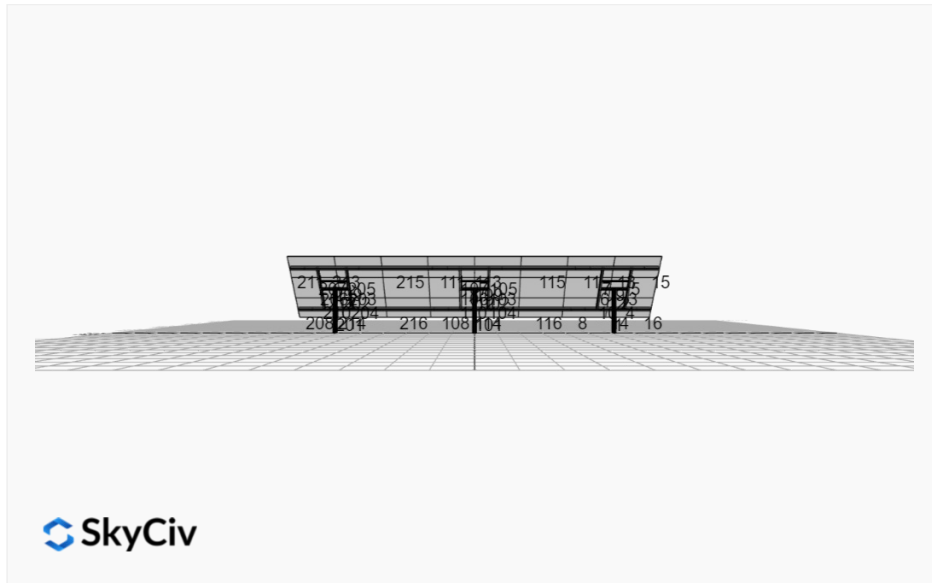
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

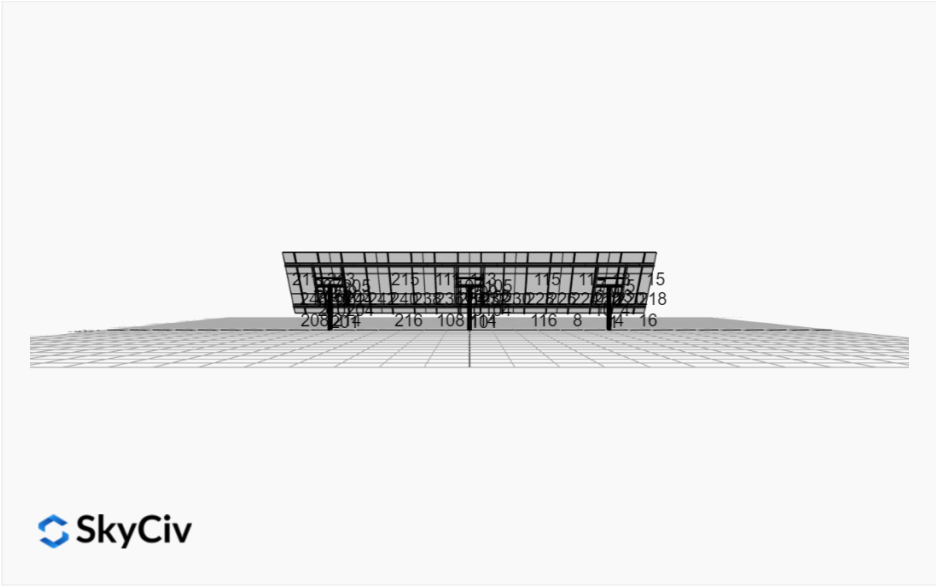
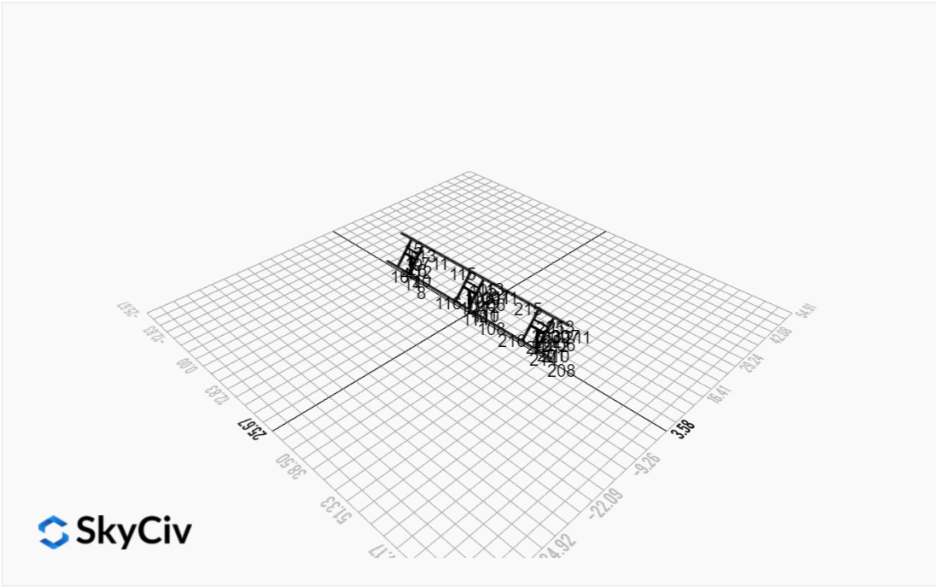
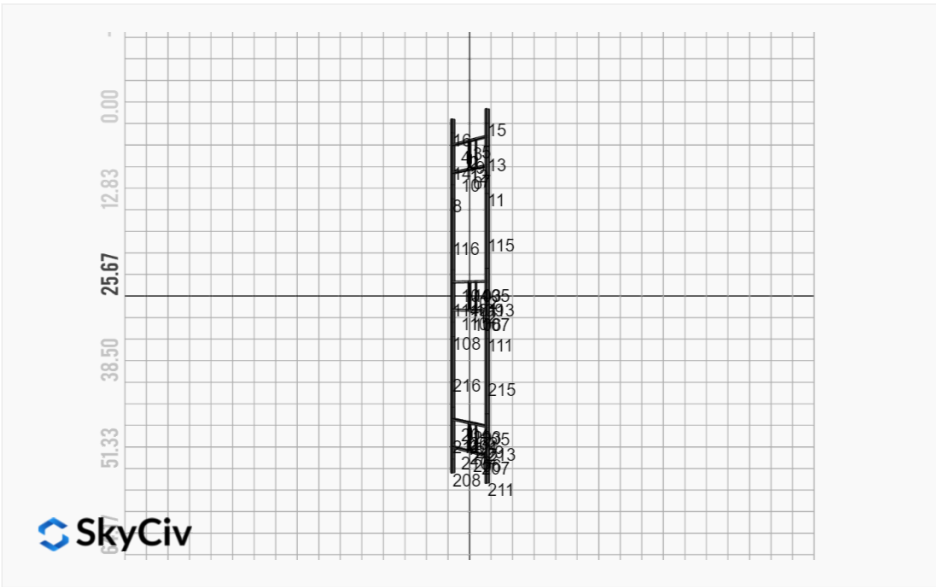
## AutoDesigner Input

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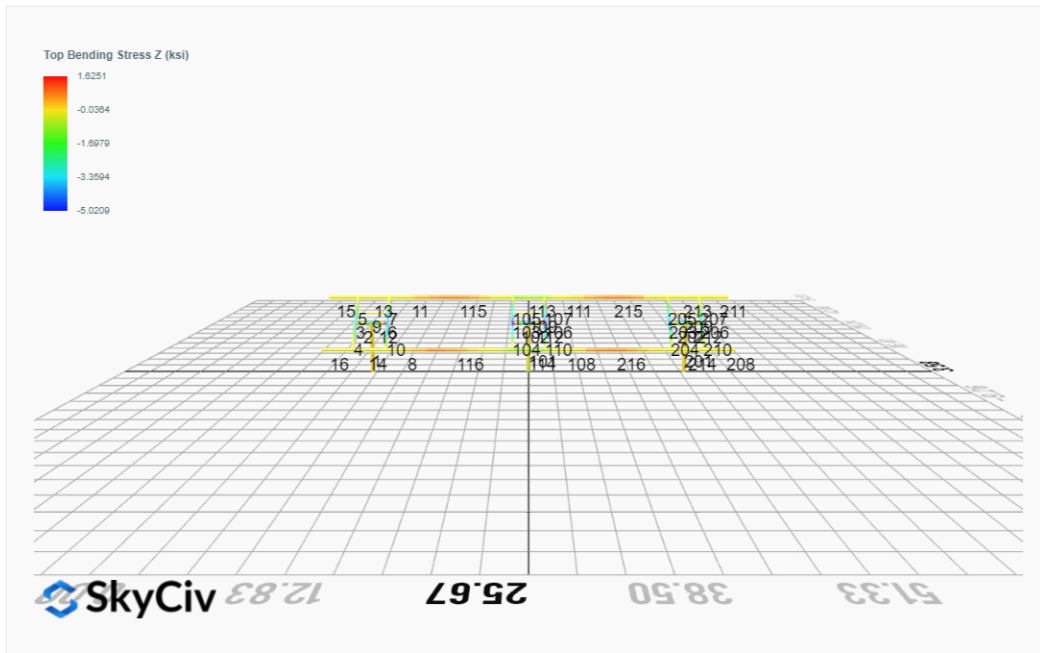
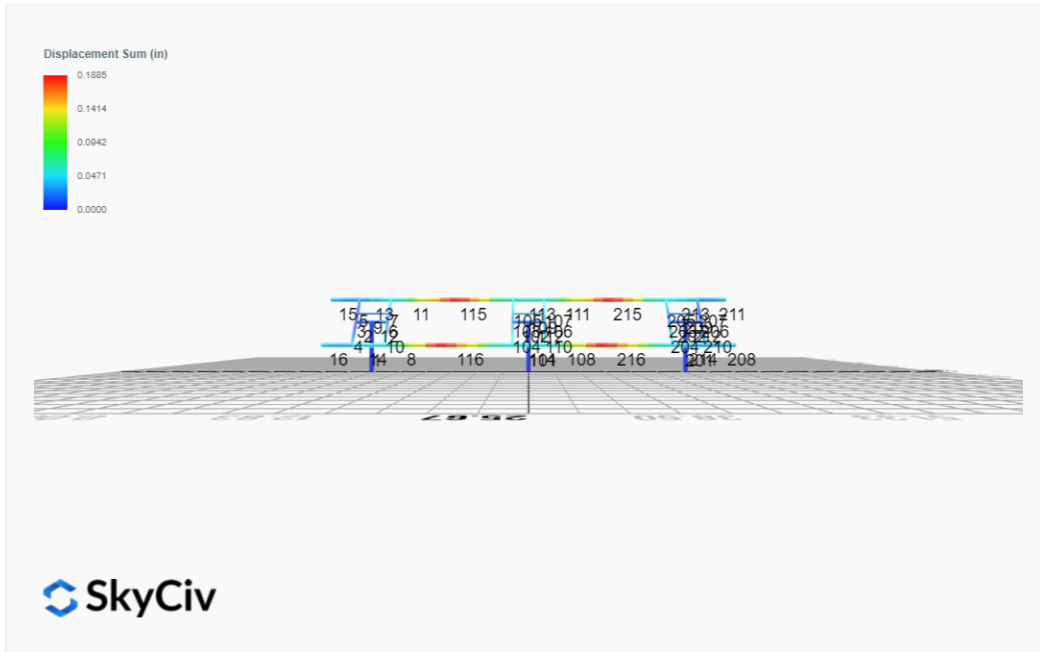
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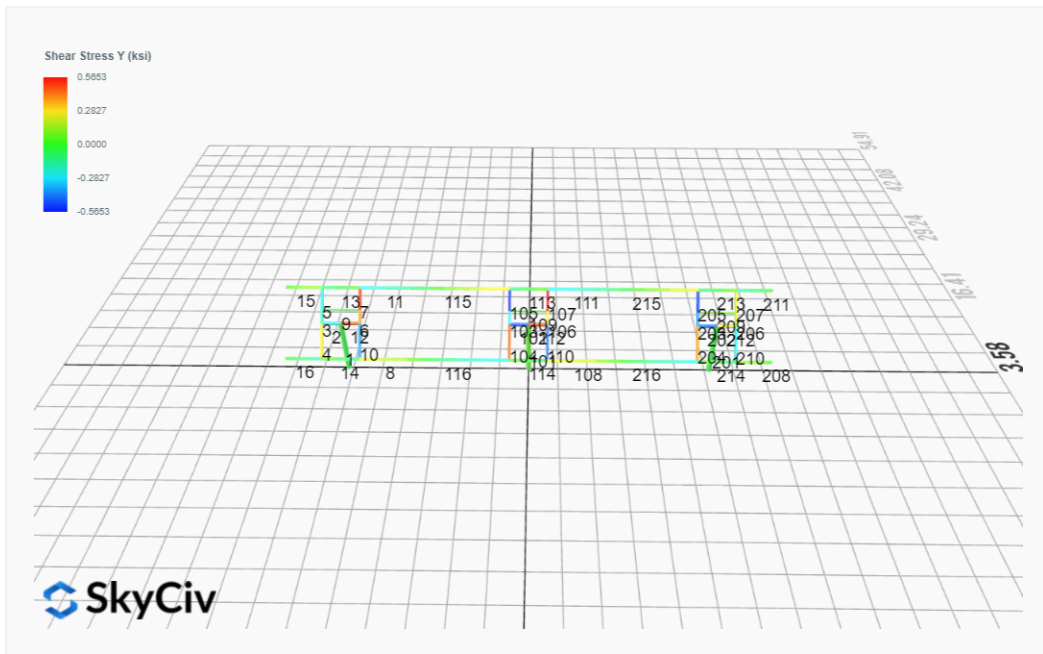
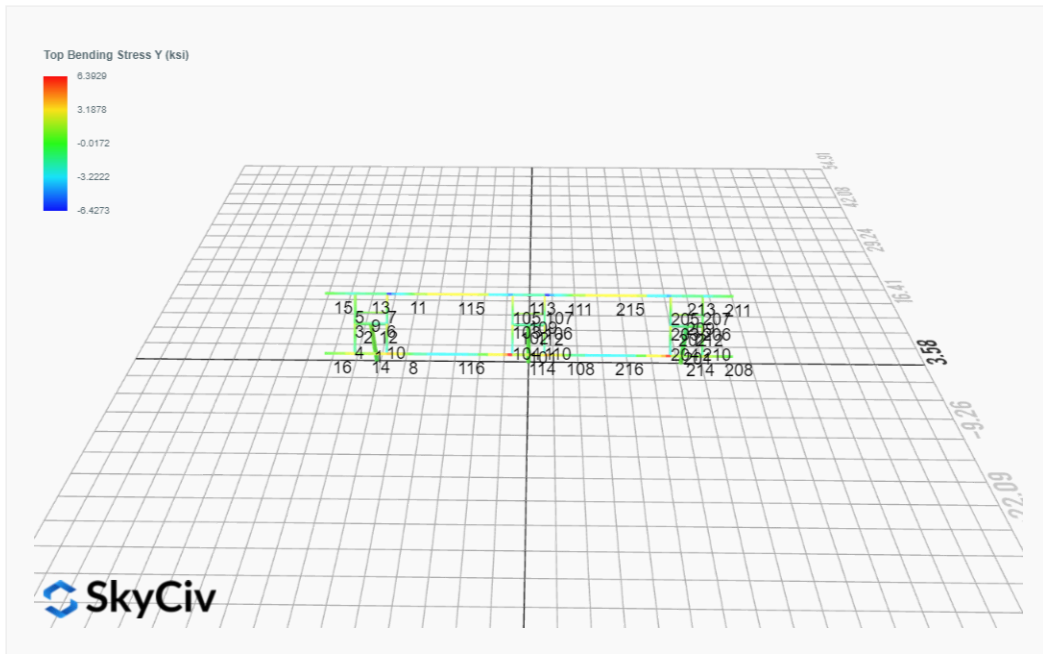
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigner are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel 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	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 2. D + L	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 3. D + (S or Lr or R)	0.0505	2.5817	0.1863	0.2632	-0.1600	-0.2694
ULS: 3. D + (S or Lr or R)	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0439	2.2805	0.1620	0.2289	-0.1391	-0.2327
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 5b. D + 0.7E	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0439	2.2805	0.1620	0.2289	-0.1391	-0.2327
ULS: 8. 0.6D + 0.7E	0.0145	0.8262	0.0535	0.0756	-0.0459	-0.0736
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6284	2.7504	0.2711	0.3414	-0.6667	10.4039
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6764	0.0037	-0.0911	-0.0877	0.5095	-10.5315
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1955	3.3106	0.2984	0.3905	-0.5817	7.6622
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0439	2.2805	0.1620	0.2289	-0.1391	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2831	1.2505	0.0267	0.0687	0.3004	-8.0394
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0439	2.2805	0.1620	0.2289	-0.1391	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2153	2.4071	0.2256	0.2876	-0.5191	7.7723
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2633	0.3470	-0.0461	-0.0343	0.3630	-7.9293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0242	1.3770	0.0892	0.1260	-0.0765	-0.1226
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6381	2.1996	0.2354	0.2910	-0.6361	10.4530
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0145	0.8262	0.0535	0.0756	-0.0459	-0.0736
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6667	-0.5471	-0.1268	-0.1381	0.5401	-10.4825
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0145	0.8262	0.0535	0.0756	-0.0459	-0.0736

### 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	4.7245
Shear X	-2.7961
Shear Z	0.4602
Moment X	0.5806
Moment Y (Twist)	1.1198
Moment Z	17.6803

### 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	3.3106
Shear X	-1.6764
Shear Z	0.2984
Moment X	0.3905
Moment Y (Twist)	0.6667
Moment Z	10.5315

## 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	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 2. D + L	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 3. D + (S or Lr or R)	-0.1010	3.3428	0.0000	-0.0000	0.0000	0.5949
ULS: 3. D + (S or Lr or R)	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0878	2.9426	0.0000	-0.0000	0.0000	0.5189

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 5b. D + 0.7E	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0878	2.9426	0.0000	-0.0000	0.0000	0.5189
ULS: 8. 0.6D + 0.7E	-0.0290	1.0451	0.0000	-0.0000	0.0000	0.1745
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.3409	3.6919	0.0000	-0.0000	0.0000	14.5035
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2449	-0.2086	0.0000	-0.0000	0.0000	-13.7247
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8072	4.4051	0.0000	-0.0000	0.0000	11.1783
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0878	2.9426	0.0000	-0.0000	0.0000	0.5189
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6322	1.4797	0.0000	-0.0000	0.0000	-9.9928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0878	2.9426	0.0000	-0.0000	0.0000	0.5189
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7678	3.2044	0.0000	-0.0000	0.0000	10.9503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6716	0.2790	0.0000	-0.0000	0.0000	-10.2208
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0484	1.7418	0.0000	-0.0000	0.0000	0.2909
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.3215	2.9952	0.0000	-0.0000	0.0000	14.3871
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0290	1.0451	0.0000	-0.0000	0.0000	0.1745
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2643	-0.9054	0.0000	-0.0000	0.0000	-13.8411
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0290	1.0451	0.0000	-0.0000	0.0000	0.1745

### 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	6.2768
Shear X	-3.9046
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0002
Moment Z	24.3872

### 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	4.4051
Shear X	-2.3409
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	14.5035

### 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	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 2. D + L	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 3. D + (S or Lr or R)	0.0505	2.5817	-0.1863	-0.2633	0.1600	-0.2694
ULS: 3. D + (S or Lr or R)	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0439	2.2805	-0.1620	-0.2290	0.1391	-0.2327
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 5b. D + 0.7E	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0439	2.2805	-0.1620	-0.2290	0.1391	-0.2327
ULS: 8. 0.6D + 0.7E	0.0145	0.8262	-0.0535	-0.0756	0.0459	-0.0736
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6284	2.7504	-0.2711	-0.3414	0.6667	10.4039
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6764	0.0037	0.0911	0.0876	-0.5095	-10.5315
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1955	3.3106	-0.2984	-0.3905	0.5817	7.6622
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0439	2.2805	-0.1620	-0.2290	0.1391	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2831	1.2505	-0.0267	-0.0687	-0.3004	-8.0394
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0439	2.2805	-0.1620	-0.2290	0.1391	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2153	2.4071	-0.2256	-0.2876	0.5192	7.7723
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2633	0.3470	0.0461	0.0342	-0.3630	-7.9293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0242	1.3770	-0.0892	-0.1260	0.0765	-0.1226
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6381	2.1996	-0.2354	-0.2910	0.6361	10.4530
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0145	0.8262	-0.0535	-0.0756	0.0459	-0.0736
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6667	-0.5471	0.1268	0.1380	-0.5401	-10.4825
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0145	0.8262	-0.0535	-0.0756	0.0459	-0.0736

### 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	4.7244
Shear X	-2.7961
Shear Z	-0.4602
Moment X	-0.5808
Moment Y (Twist)	1.1201
Moment Z	17.6806

### 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	3.3106
Shear X	-1.6764
Shear Z	-0.2984
Moment X	-0.3905
Moment Y (Twist)	0.6667
Moment Z	10.5315

## Project Details

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

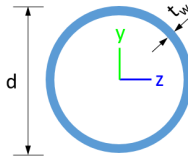


## Design Input Information

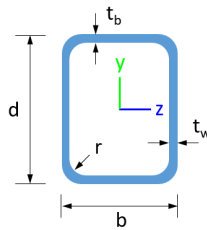
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
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Design Materials			
ID	E (ksi)	$F_y$ (ksi)	$F_u$ (ksi)
1	29000	50	65

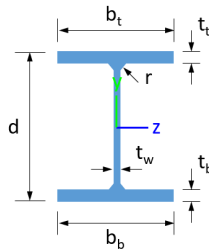
### Section Dimensions



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> )
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101	251.10	175.05	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	6.26	29.14	16.61
104	79.65	72.01	10.99	6.26	29.14	16.61
105	79.65	73.44	10.99	6.26	29.14	16.61
106	79.65	74.02	10.99	6.26	29.14	16.61
107	79.65	73.44	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	17.92	6.45	30.09	45.74
114	120.60	84.03	18.16	6.45	30.09	45.74
115	120.60	68.63	15.30	6.45	30.09	45.74
116	120.60	68.63	15.91	6.45	30.09	45.74
201	251.16	175.05	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	6.26	29.14	16.61
204	79.65	72.01	10.99	6.26	29.14	16.61
205	79.65	73.44	10.99	6.26	29.14	16.61
206	79.65	74.02	10.99	6.26	29.14	16.61
207	79.65	73.44	10.99	6.26	29.14	16.61
208	120.60	96.18	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	6.26	29.14	16.61
211	120.60	96.18	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.03	20.29	6.45	30.09	45.74
214	120.60	84.03	20.00	6.45	30.09	45.74
215	120.60	68.63	14.88	6.45	30.09	45.74
216	120.60	68.63	14.89	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	0.027	0.418	0.054	0.037	0.006	0.439	#13	0.351	Not Required	Pass
2	0.002	0.153	0.131	0.040	0.028	0.281	#13	0.052	Not Required	Pass
3	0.007	0.341	0.025	0.033	0.002	0.344	#13	0.044	Not Required	Pass
4	0.005	0.340	0.090	0.034	0.018	0.421	#13	0.078	Not Required	Pass
5	0.006	0.211	0.056	0.034	0.011	0.218	#13	0.073	Not Required	Pass
6	0.011	0.499	0.133	0.052	0.029	0.592	#13	0.044	Not Required	Pass
7	0.012	0.309	0.192	0.050	0.038	0.344	#13	0.073	Not Required	Pass
8	0.003	0.091	0.094	0.028	0.011	0.123	#13	0.088	Not Required	Pass
9	0.004	0.039	0.071	0.003	0.004	0.093	#13	0.198	Not Required	Pass
10	0.012	0.474	0.178	0.048	0.031	0.502	#13	0.078	Not Required	Pass
11	0.004	0.086	0.096	0.030	0.011	0.114	#13	0.088	Not Required	Pass
12	0.001	0.291	0.200	0.067	0.040	0.492	#13	0.052	Not Required	Pass
13	0.006	0.077	0.252	0.039	0.014	0.300	#21	0.265	Not Required	Pass
14	0.004	0.071	0.249	0.037	0.014	0.278	#21	0.177	Not Required	Pass
15	0.000	0.014	0.025	0.011	0.004	0.036	#21	Not Required	Not Required	Pass

16	0.000	0.014	0.025	0.011	0.004	0.036	#21	Not Required	Not Required	Pass
101	0.036	0.577	0.000	0.052	0.000	0.594	#13	0.351	Not Required	Pass
102	0.002	0.306	0.243	0.072	0.046	0.549	#13	0.034	Not Required	Pass
103	0.011	0.571	0.082	0.057	0.012	0.623	#13	0.044	Not Required	Pass
104	0.010	0.589	0.185	0.059	0.032	0.672	#13	0.078	Not Required	Pass
105	0.011	0.354	0.193	0.057	0.038	0.385	#13	0.073	Not Required	Pass
106	0.011	0.571	0.082	0.057	0.012	0.623	#13	0.044	Not Required	Pass
107	0.011	0.354	0.193	0.057	0.038	0.385	#13	0.073	Not Required	Pass
108	0.003	0.044	0.099	0.037	0.011	0.131	#21	0.088	Not Required	Pass
109	0.011	0.036	0.051	0.001	0.000	0.091	#13	0.198	Not Required	Pass
110	0.010	0.589	0.185	0.059	0.032	0.672	#13	0.078	Not Required	Pass
111	0.004	0.061	0.100	0.035	0.011	0.119	#21	0.088	Not Required	Pass
112	0.002	0.306	0.243	0.072	0.046	0.549	#13	0.034	Not Required	Pass
113	0.006	0.148	0.257	0.044	0.014	0.374	#21	0.265	Not Required	Pass
114	0.005	0.185	0.256	0.046	0.014	0.393	#21	0.265	Not Required	Pass
115	0.007	0.193	0.144	0.035	0.011	0.301	#21	0.439	Not Required	Pass
116	0.003	0.182	0.145	0.037	0.011	0.290	#21	0.439	Not Required	Pass
201	0.027	0.418	0.054	0.037	0.006	0.439	#13	0.351	Not Required	Pass
202	0.001	0.291	0.200	0.067	0.040	0.492	#13	0.052	Not Required	Pass
203	0.011	0.499	0.133	0.052	0.029	0.592	#13	0.044	Not Required	Pass
204	0.012	0.474	0.178	0.048	0.031	0.502	#13	0.078	Not Required	Pass
205	0.012	0.309	0.192	0.050	0.038	0.344	#13	0.073	Not Required	Pass
206	0.007	0.341	0.025	0.033	0.002	0.344	#13	0.044	Not Required	Pass
207	0.006	0.211	0.056	0.034	0.011	0.218	#13	0.073	Not Required	Pass
208	0.000	0.014	0.025	0.011	0.004	0.036	#21	Not Required	Not Required	Pass
209	0.004	0.039	0.071	0.003	0.004	0.093	#13	0.198	Not Required	Pass
210	0.005	0.340	0.090	0.034	0.018	0.421	#13	0.078	Not Required	Pass
211	0.000	0.014	0.025	0.011	0.004	0.036	#21	Not Required	Not Required	Pass
212	0.002	0.153	0.131	0.040	0.028	0.281	#13	0.052	Not Required	Pass
213	0.006	0.077	0.252	0.039	0.014	0.300	#21	0.177	Not Required	Pass
214	0.004	0.071	0.249	0.037	0.014	0.278	#21	0.265	Not Required	Pass
215	0.007	0.204	0.145	0.030	0.011	0.307	#21	0.439	Not Required	Pass
216	0.003	0.198	0.144	0.028	0.011	0.297	#21	0.439	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
$F_y$	Specified minimum yield stress
$F_u$	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
$I_{yp}$	Moment of inertia about the Y axes
$I_{zp}$	Moment of inertia about the Z axes
$I_w$	Warping constant
$S_{yp}$	Plastic section modulus about the Y axis
$S_{zp}$	Plastic section modulus about the Z axis
KL	Effective length
$C_b$	Buckling modification factor (from all load combinations)
$L_b$	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
$P_n$	Nominal axial strength (tension/compression)

$M_n$	Nominal flexural strength (about Z/Y axis)
$V_n$	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
$M_z$	Design ratio in case of bending about Z axis
$M_y$	Design ratio in case of bending about Y axis
$V_y$	Design ratio in case of shear along Y axis
$V_z$	Design ratio in case of shear along Z axis
(P, $M_z$ , $M_y$ )	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS
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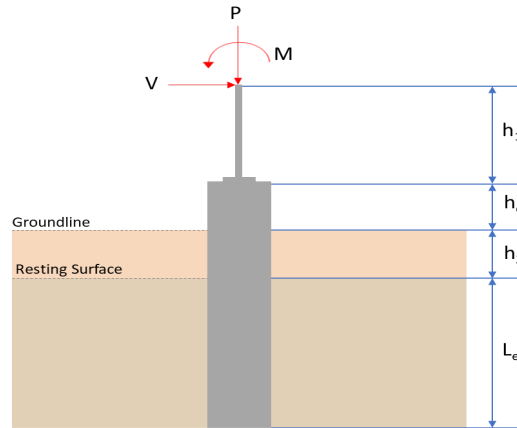
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 6.25$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	3.311	4.724
$V_x$ (kip)	-1.676	-2.796
$V_z$ (kip)	0.298	0.460
$M_x$ (kipft)	0.390	0.581
$M_z$ (kipft)	10.532	17.680

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.676 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.55867 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(10.532 \text{ kipft}) + ((-1.676 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.5107 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3952 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.298 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.099333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.39 \text{ kipft}) + ((0.298 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.13 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.7103 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.3952 \text{ ft}), (3.7103 \text{ ft})]$$

$$L_{e,req} = 5.395 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.395 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.8632$$

Status: **PASS**  
Ratio: **0.860**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(3.311 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.46841 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46841 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23421$$

Status: **PASS**  
Ratio: **0.230**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.55867 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.5107 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.5107 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.5107 \text{ kipft/ft})) + (4 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3743 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.5107 \text{ kipft/ft})) + (3 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.5107 \text{ kipft/ft})) + (2 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.10817 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (3.5107 \text{ kipft/ft})) + ((-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85164 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3743 \text{ ft})}{2}$$

$$p_a = 0.32807 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.10817 \text{ kip/ft}^2)}{(0.32807 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.3297$$

$p_s$  - 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

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

$$\text{Ratio} = \frac{(0.85164 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90841$$

Status: **PASS**  
Ratio: **0.330**

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

**Considering z-direction:**

$H_o = 0.099333 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.13 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.13 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.099333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.13 \text{ kipft/ft})) + (4 \times (0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.563 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.13 \text{ kipft/ft})) + (3 \times (0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.13 \text{ kipft/ft})) + (2 \times (0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.10491 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.13 \text{ kipft/ft})) + ((0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.21253 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.563 \text{ ft})}{2}$$

$$p_a = 0.34223 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.10491 \text{ kip/ft}^2)}{(0.34223 \text{ kip/ft}^2)}$$

$$(0.07220 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.30655$$

$p_s$  - 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

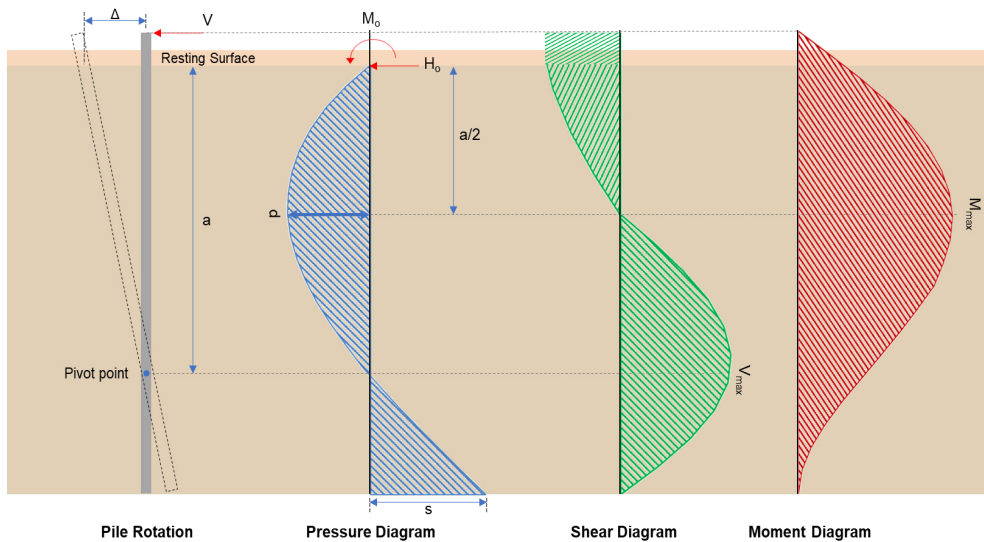
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.21253 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.2267$$

Status: **PASS**  
Ratio: **0.310**

Status: **PASS**  
Ratio: **0.230**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.796 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.932 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(17.68 \text{ kipft}) + ((-2.796 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8933 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.8933 \text{ kipft/ft})}{(-0.932 \text{ kip/ft})}$$

$$E = 6.3233 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.8933 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.932 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.8933 \text{ kipft/ft})) + (4 \times (-0.932 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3735 \text{ ft}$$

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

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

$$V_{max} = ((-0.932 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.3233 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3735 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6.3233 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3735 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.8521 \text{ kip}$$

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

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

$$M_{max} = ((-0.932 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(6.3233 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.3233 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (6.3233 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.838 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.46 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.15333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.581 \text{ kipft}) + ((0.46 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19367 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.19367 \text{ kipft/ft})}{(0.15333 \text{ kip/ft})}$$

$$E = 1.263 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.19367 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.15333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19367 \text{ kipft/ft})) + (4 \times (0.15333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5663 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = ((0.15333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.5663 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.5663 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.47513 \text{ kip}$$

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

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

$$M_{max} = ((0.15333 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(1.263 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2309 \text{ kipft}$$

### 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,

Table 22.4.2.1

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(4.724 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.226 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.226 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 6$$

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

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

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

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

Ratio - Capacity

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

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

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</math></p> <p><math>s_{rebar} = Max [1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]</math></p> <p><math>s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \text{ kip}</math></p> <p>Ratio - Capacity</p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(4.724 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0037674</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 36 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 4.724 \text{ kip} \rightarrow 4724 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(4724 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.24 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.24 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.24 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.24 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.717 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 6.8521 \text{ kip}$  - Maximum shear force in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(6.8521 \text{ kip})}{(73.717 \text{ kip})}$$

$$Ratio = 0.092952$$

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

**Considering z-direction:**

$V_{max} = 0.47513 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.47513 \text{ kip})}{(73.717 \text{ kip})}$$

$$Ratio = 0.0064454$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 19.838 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.838 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.31983$$

Status: **PASS**  
Ratio: **0.320**

**Considering z-direction:**

$M_{max} = 1.2309 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(1.2309 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.019845$$

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

REFERENCES	CALCULATIONS	RESULTS
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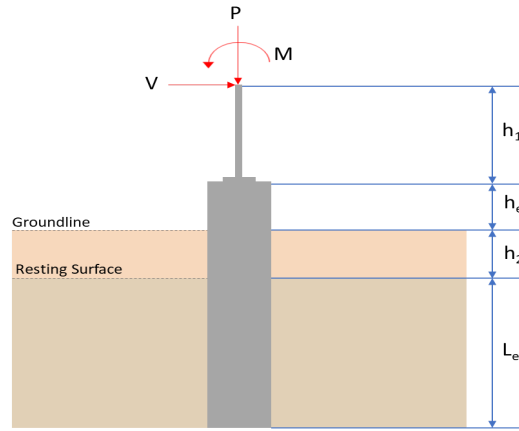
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 6.25$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	3.311	4.724
$V_x$ (kip)	-1.676	-2.796
$V_z$ (kip)	-0.298	-0.460
$M_x$ (kipft)	-0.391	-0.581
$M_z$ (kipft)	10.532	17.681

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.676 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.55867 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(10.532 \text{ kipft}) + ((-1.676 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 3.5107 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3952 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.298 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.099333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.391 \text{ kipft}) + ((-0.298 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.13033 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4341 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.3952 \text{ ft}), (1.4341 \text{ ft})]$$

$$L_{e,req} = 5.395 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.395 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.8632$$

Status: **PASS**  
Ratio: **0.860**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(3.311 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.46841 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46841 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23421$$

Status: **PASS**  
Ratio: **0.230**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 2.0833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.55867 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.5107 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.5107 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.5107 \text{ kipft/ft})) + (4 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3743 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.5107 \text{ kipft/ft})) + (3 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.5107 \text{ kipft/ft})) + (2 \times (-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.10817 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (3.5107 \text{ kipft/ft})) + ((-0.55867 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85164 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3743 \text{ ft})}{2}$$

$$p_a = 0.32807 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.10817 \text{ kip/ft}^2)}{(0.32807 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.3297$$

$p_s$  - 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

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

$$\text{Ratio} = \frac{(0.85164 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90841$$

Status: **PASS**  
Ratio: **0.330**

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

**Considering z-direction:**

$H_o = -0.099333 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.13033 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.13033 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.099333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.13033 \text{ kipft/ft})) + (4 \times (-0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5628 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.13033 \text{ kipft/ft})) + (3 \times (-0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.13033 \text{ kipft/ft})) + (2 \times (-0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.063766 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.13033 \text{ kipft/ft})) + ((-0.099333 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.086901 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5628 \text{ ft})}{2}$$

$$p_a = 0.34221 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.063766 \text{ kip/ft}^2)}{(0.34221 \text{ kip/ft}^2)}$$

$$(0.07221 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.18634$$

$p_s$  - 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

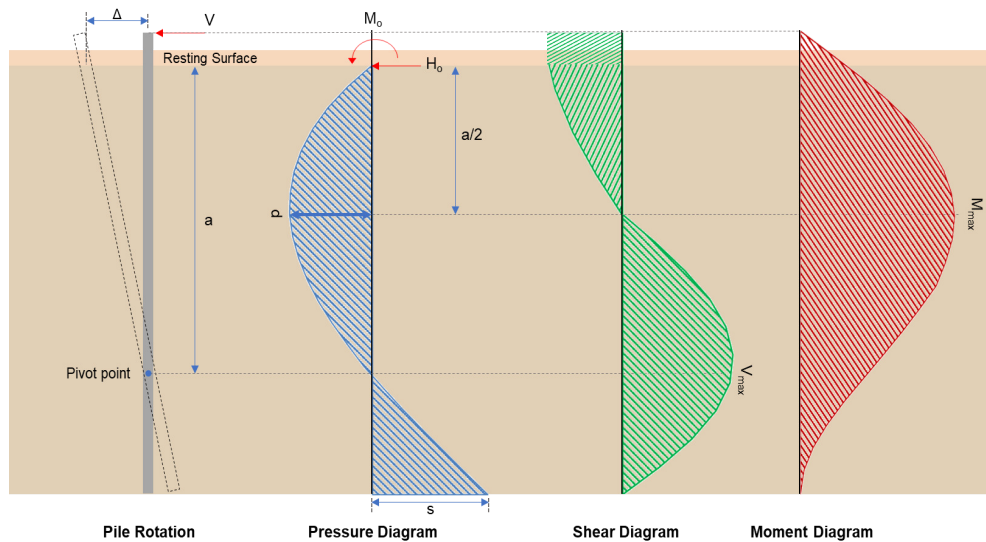
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.086901 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.092694$$

Status: **PASS**  
Ratio: **-0.190**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.796 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.932 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(17.681 \text{ kipft}) + ((-2.796 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8937 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.8937 \text{ kipft/ft})}{(-0.932 \text{ kip/ft})}$$

$$E = 6.3237 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.8937 \text{ kip/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.932 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.8937 \text{ kip/ft})) + (4 \times (-0.932 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3735 \text{ ft}$$

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

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

$$V_{max} = ((-0.932 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.3237 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3735 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6.3237 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3735 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.8524 \text{ kip}$$

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

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

$$M_{max} = ((-0.932 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(6.3237 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.3237 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (6.3237 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3735 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.839 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.46 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.15333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.581 \text{ kipft}) + ((-0.46 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19367 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.19367 \text{ kipft/ft})}{(-0.15333 \text{ kip/ft})}$$

$$E = 1.263 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.19367 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.15333 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19367 \text{ kipft/ft})) + (4 \times (-0.15333 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5663 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.15333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.5663 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.5663 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.47513 \text{ kip}$$

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

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

$$M_{max} = ((-0.15333 \text{ kip/ft}) \times (36 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(1.263 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (1.263 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.5663 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2309 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(4.724 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.226 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.226 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 6$$

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

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

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

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

Ratio - Capacity

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

$$(1.8322 \text{ in}^2)$$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</math></p> <p><math>s_{rebar} = Max [1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]</math></p> <p><math>s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \text{ kip}</math></p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(4.724 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0037674</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 36 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 4.724 \text{ kip} \rightarrow 4724 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(4724 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.24 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.24 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.24 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.24 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.717 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 6.8524 \text{ kip}$  - Maximum shear force in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(6.8524 \text{ kip})}{(73.717 \text{ kip})}$$

$$Ratio = 0.092956$$

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

**Considering z-direction:**

$V_{max} = 0.47513 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.47513 \text{ kip})}{(73.717 \text{ kip})}$$

$$Ratio = 0.0064454$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 19.839 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.839 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.31985$$

Status: **PASS**  
Ratio: **0.320**

**Considering z-direction:**

$M_{max} = 1.2309 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(1.2309 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.019845$$

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

REFERENCES	CALCULATIONS	RESULTS
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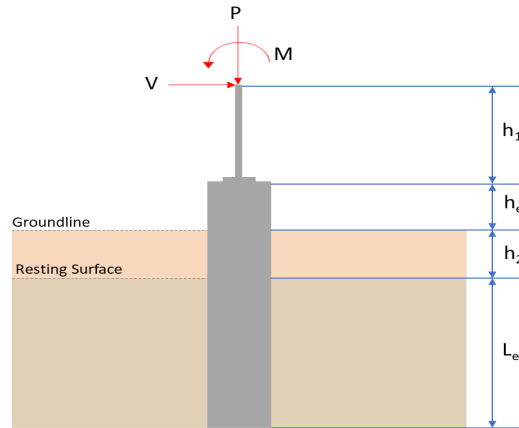
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 6.75$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.405	6.277
$V_x$ (kip)	-2.341	-3.905
$V_z$ (kip)	0.000	0.000
$M_x$ (kipft)	0.000	0.000
$M_z$ (kipft)	14.503	24.387

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.341 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.78033 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(14.503 \text{ kipft}) + ((-2.341 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.8343 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.7177 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

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

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.7177 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.718 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.718 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.84711$$

Status: **PASS**  
Ratio: **0.850**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(4.405 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.62318 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62318 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.31159$$

Status: **PASS**  
Ratio: **0.310**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.78033 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.8343 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.8343 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.78033 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.8343 \text{ kipft/ft})) + (4 \times (-0.78033 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7367 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (4.8343 \text{ kipft/ft})) + (3 \times (-0.78033 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.8343 \text{ kipft/ft})) + (2 \times (-0.78033 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.081439 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (4.8343 \text{ kipft/ft})) + ((-0.78033 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.91047 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7367 \text{ ft})}{2}$$

$$p_a = 0.35525 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.081439 \text{ kip/ft}^2)}{(0.35525 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.22924$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

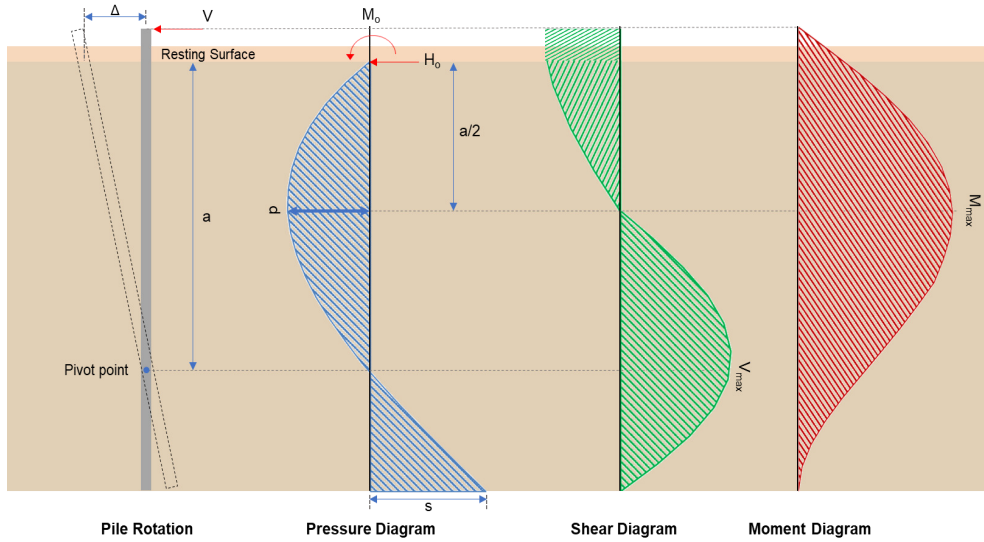
$$\dots (0.91047 \text{ kip/ft}^2)$$

Status: **PASS**  
Ratio: **0.230**

$$\text{Ratio} = \frac{\dots}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89923$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.905 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3017 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(24.387 \text{ kipft}) + ((-3.905 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.129 \text{ kipft/ft}$$

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

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

$$E = \frac{(8.129 \text{ kipft/ft})}{(-1.3017 \text{ kip/ft})}$$

$$E = 6.2451 \text{ ft}$$

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

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

$$a = \frac{(4 \times (8.129 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-1.3017 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.129 \text{ kipft/ft})) + (4 \times (-1.3017 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7356 \text{ ft}$$

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

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

$$V_{max} = ((-1.3017 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6.2451 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.7356 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (6.2451 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.7356 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.9741 \text{ kip}$$

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

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

$$M_{max} = ((-1.3017 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(6.2451 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.7356 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6.2451 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.7356 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (6.2451 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.7356 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.915 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(6.277 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.177 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.177 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 6$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

25.2.3

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

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

<p>25.7.2.2 25.7.2.1</p>	$s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b> Since longitudinal reinforcement is <math>\leq</math> No. 10: Use #3(0.375 in) <math>s_{ties}</math> - Maximum center-to-center spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p>Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$ $\phi P_N = 1253.9 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(6.277 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0050059$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2  22.5.5.1.3  22.5.5.1.1  22.5.5.1.1(a)</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b> <math>b_w = 36 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>, <math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>, <math>P = 6.277 \text{ kip} \rightarrow 6277 \text{ lbf}</math>, <math>V_{c,a}</math> - Shear strength of concrete (a)</p> $V_{c,a} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(6277 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.504 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (75.504 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.504 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN} [(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.504 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.888 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.9741 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.9741 \text{ kip})}{(73.888 \text{ kip})}$$

$$\text{Ratio} = 0.12145$$

Status: **PASS**  
 Ratio: 0.12145

**Flexural Strength (ACI 318-19, LRFD)** $S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

 $M_n$  shall be the lesser of: $\phi M_{n,1}$ 

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

 $\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:** $M_{max} = 27.915 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(27.915 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.45005$$