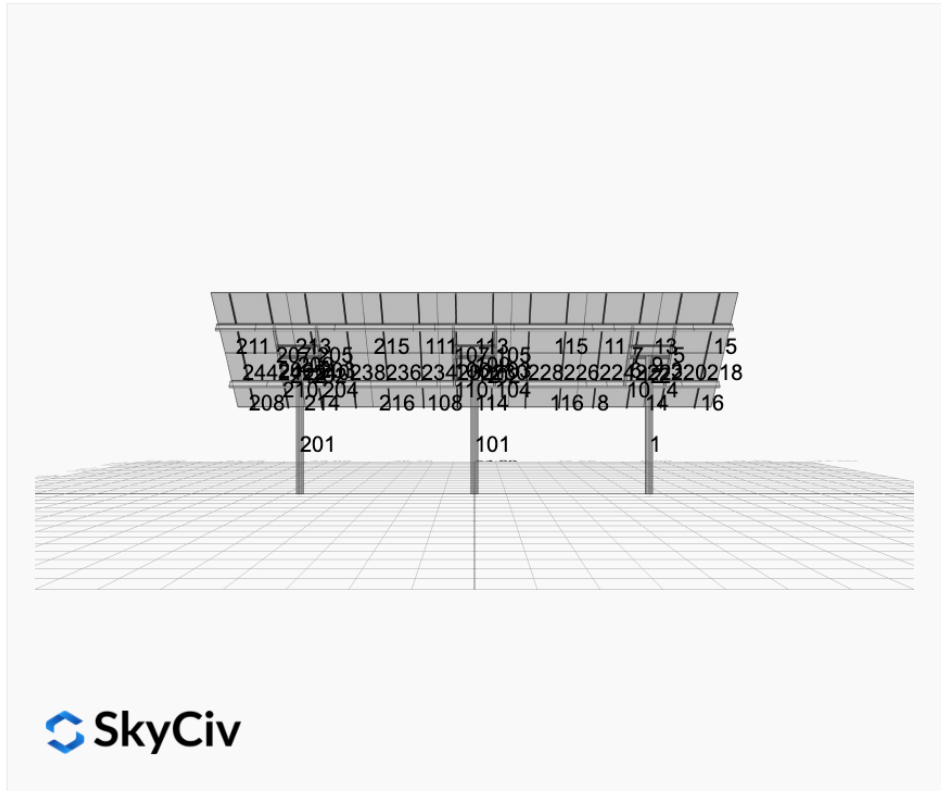


# Project Details



**Project Name:** MTSOLAR\_0A4A980B6GB54 4x7 - V1Jb  
**Date:** Thu Jul 31 2025  
**Location:** 128 School St, Gorham, ME 04038, USA  
**Number of Modules:** 28  
**Unique ID:** 3P-17-8TOP-HD-45-L-4Hx7W-8I3K  
**Number of Poles:** 3  
**Dealer:** \_\_\_\_\_  
**Date Sold:** \_\_\_\_\_



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	48.77 ft
Winter Tilt Angle	45
Front Edge Clearance	8 ft

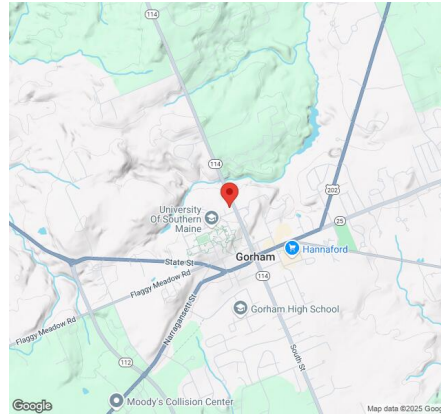
## MT Solar Bill of Materials (3P-17-8TOP-HD-45-L-4Hx7W-8I3K)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	7

## Rail Bill of Materials

Part	Qty
Rails (180in)	14
Rail Attachment	28
Module Mid Clamp	42
Module End Clamp	28
Ground Lug	7

## Site Details:



**Site Address:** 128 School St, Gorham, ME 04038, USA

### Array Specification

<b>Duty Classification:</b>	HD
<b>Module Width:</b>	44.60 in
<b>Module Length:</b>	82.60in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	7
<b>Total Number of Modules:</b>	28
<b>Winter Tilt Angle:</b>	45
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	18.63 ft
<b>Total Frame Length:</b>	49.00 ft
<b>Module Info/Notes:</b>	SILAB 530 XM
<b>Array Dimensions N/S:</b>	15.03 ft
<b>Array Dimensions E/W:</b>	48.77 ft
<b>Rail Length:</b>	180.40 in
<b>Rail Spacing:</b>	3.48 ft

### Support Specifications

<b>Pole Size:</b>	8in Pipe Sch 80
<b>Pole Length above Grade:</b>	13.32 ft
<b>Number of Poles:</b>	3
<b>Pole Spacing:</b>	17 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 8.25 ft Pile 2: 8.25 ft Pile 3: 8.25 ft
<b>Foundation Volume:</b>	14.667 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	128 School St, Gorham, ME 04038, USA
<b>Wind Speed:</b>	104 mph
<b>Snow Load:</b>	60 psf

### **Design Disclaimer**

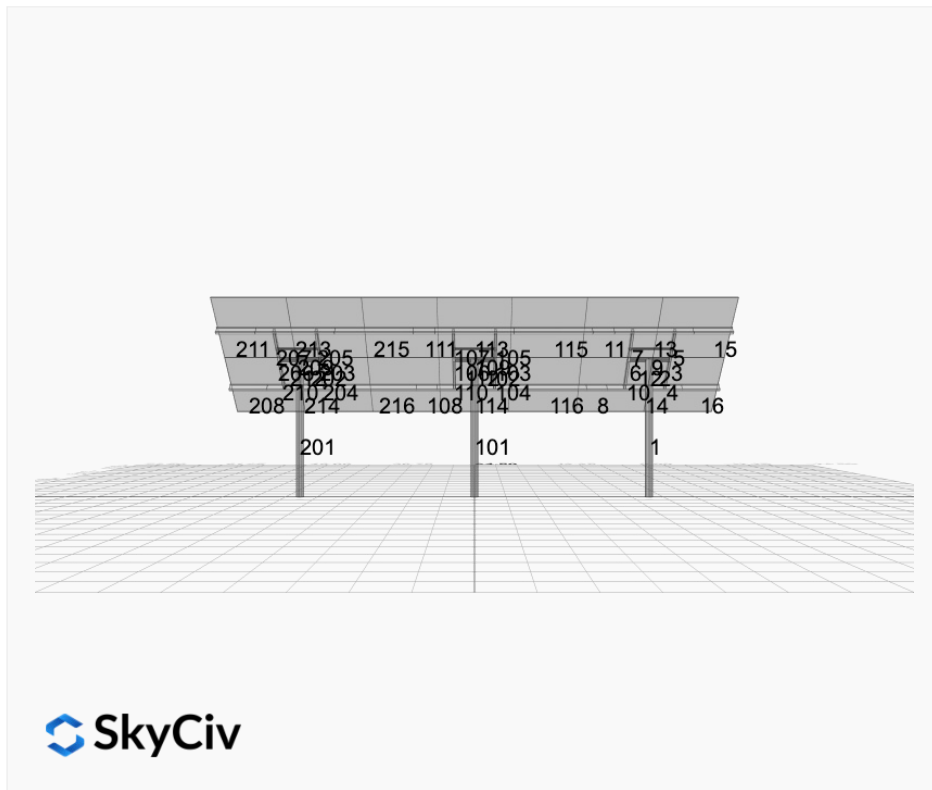
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

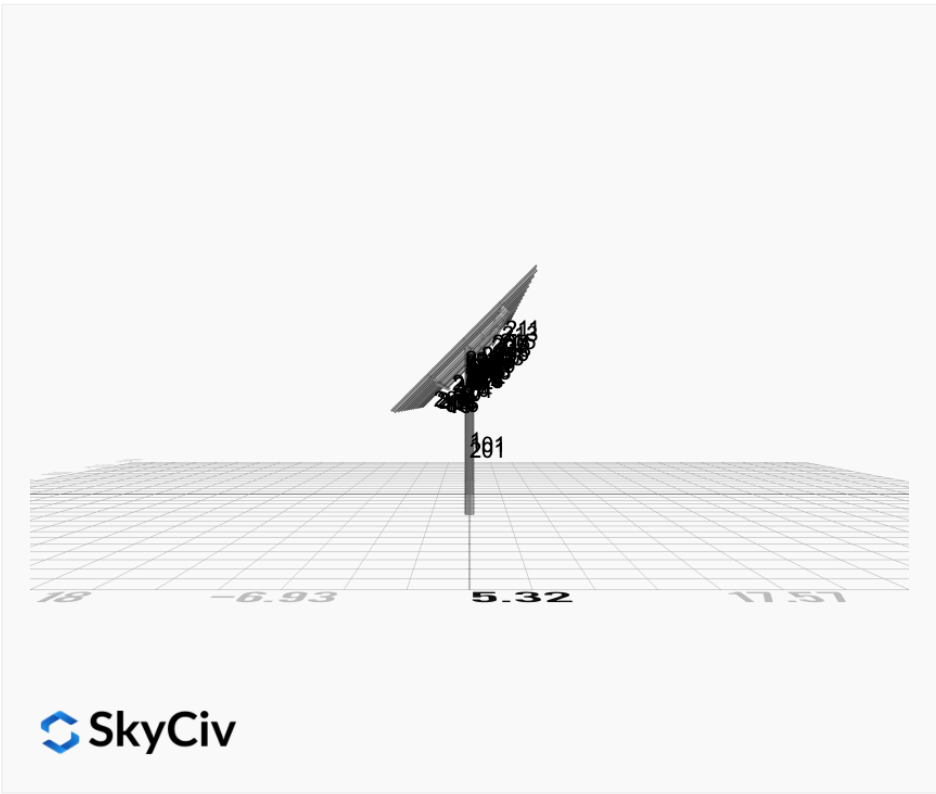
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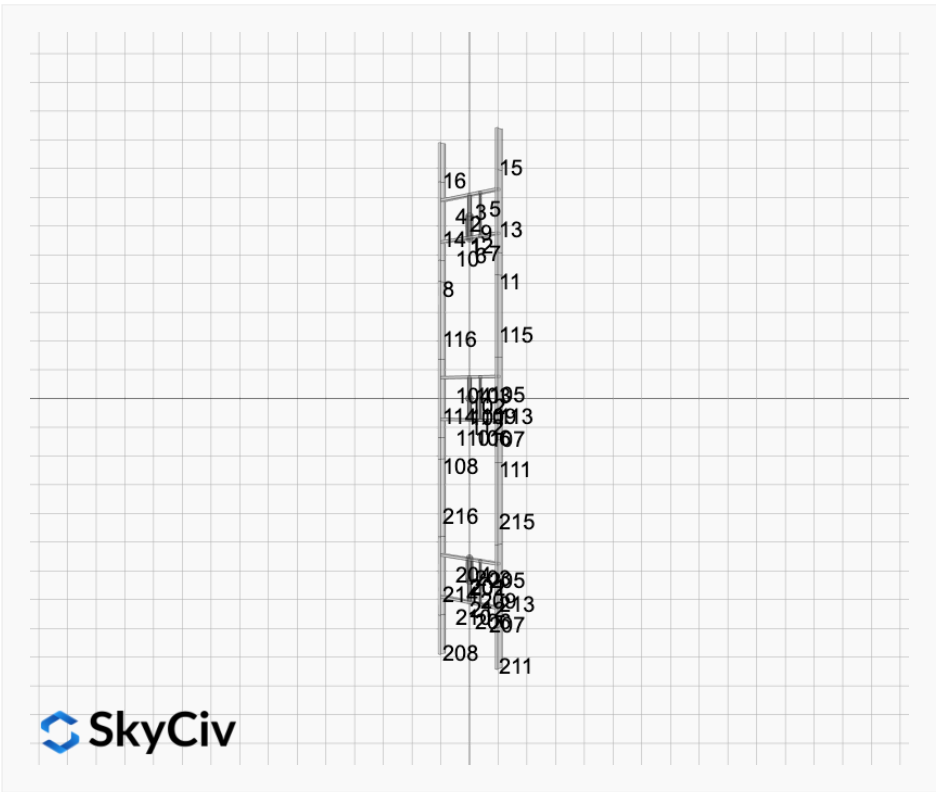
## Design Notes:

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

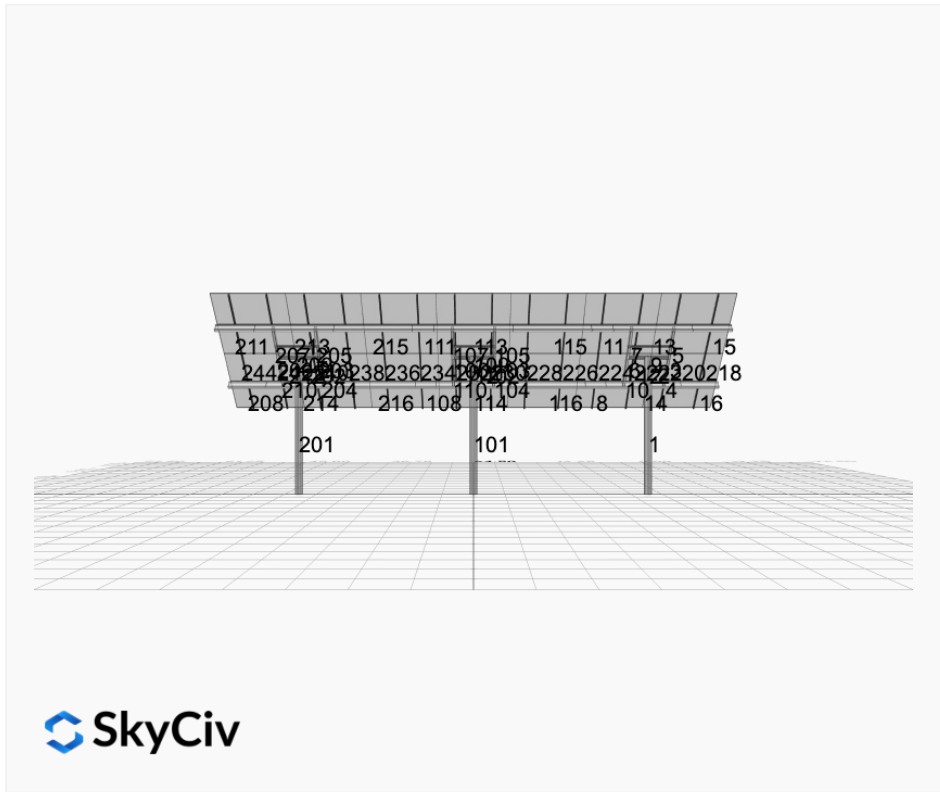
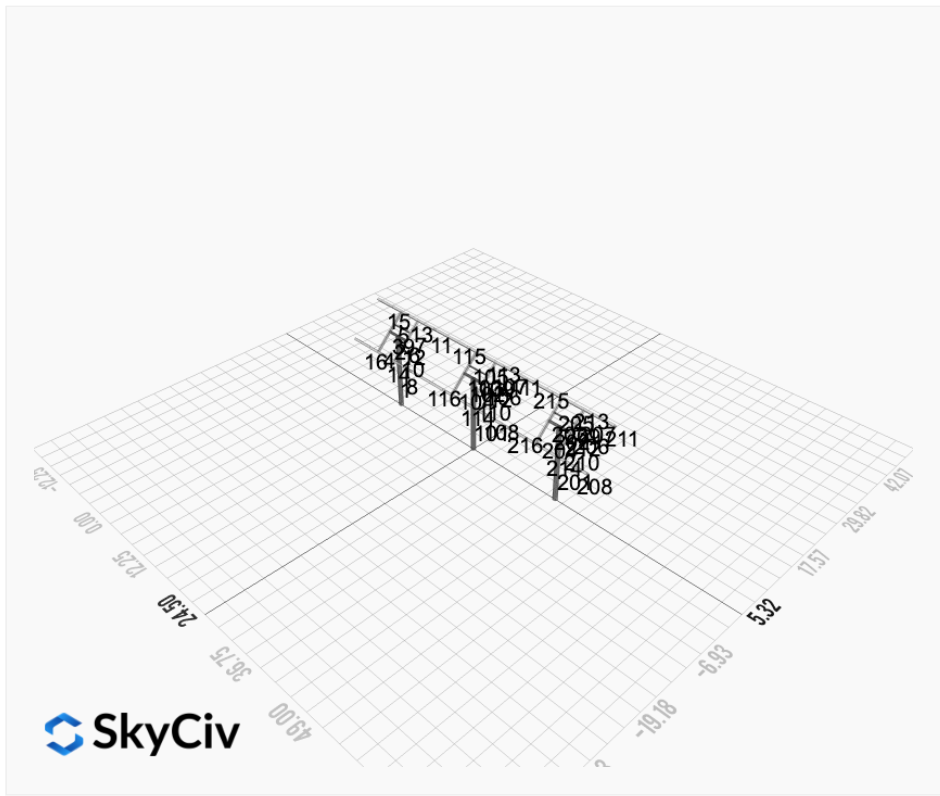




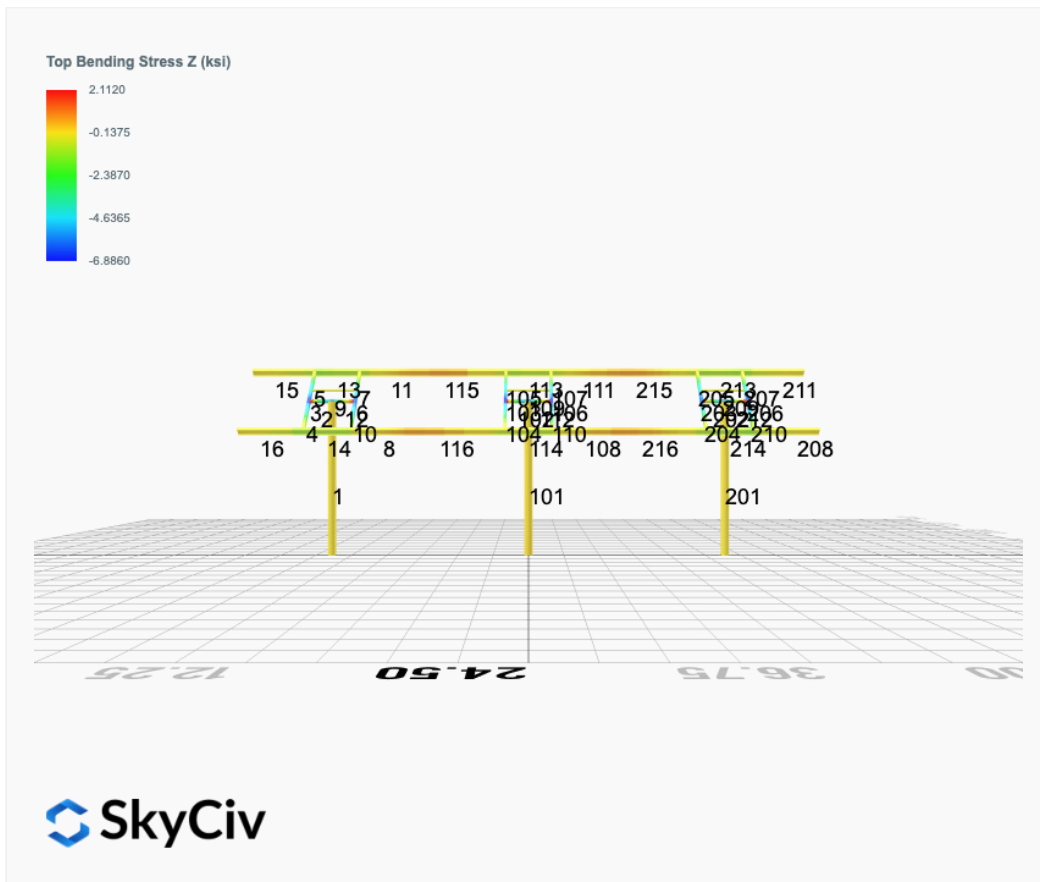
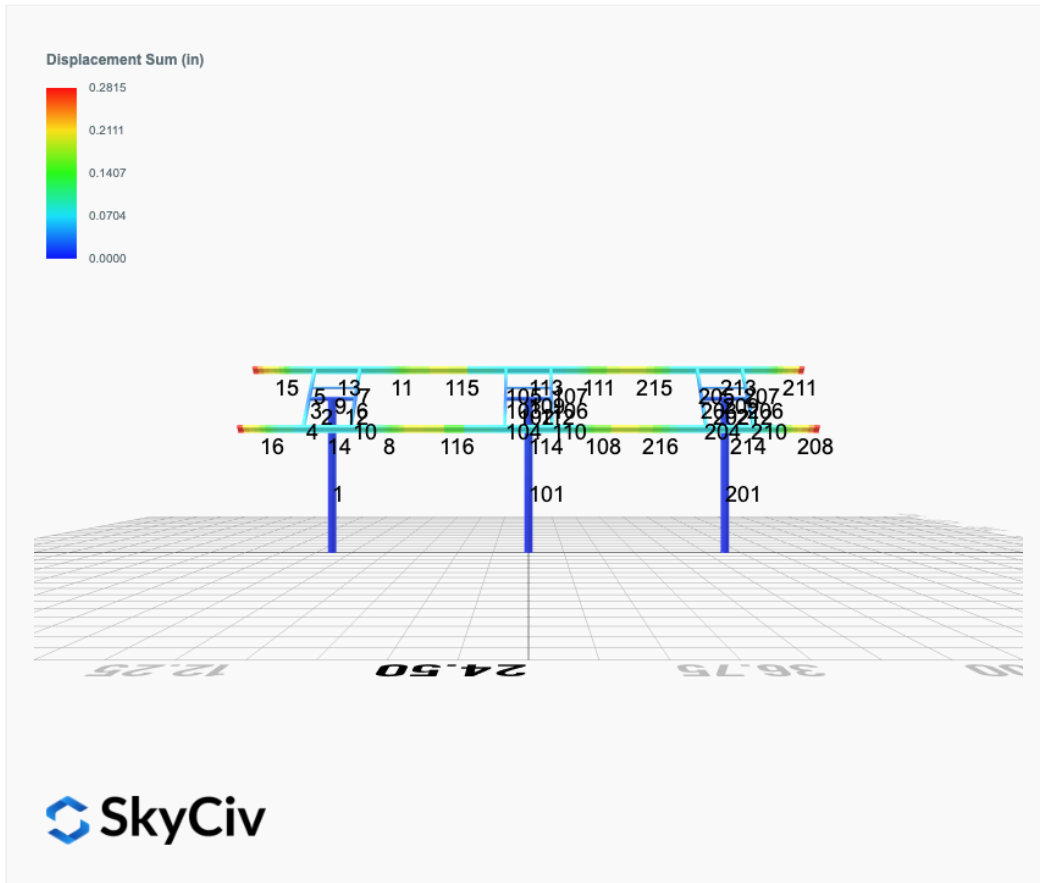
 SkyCiv



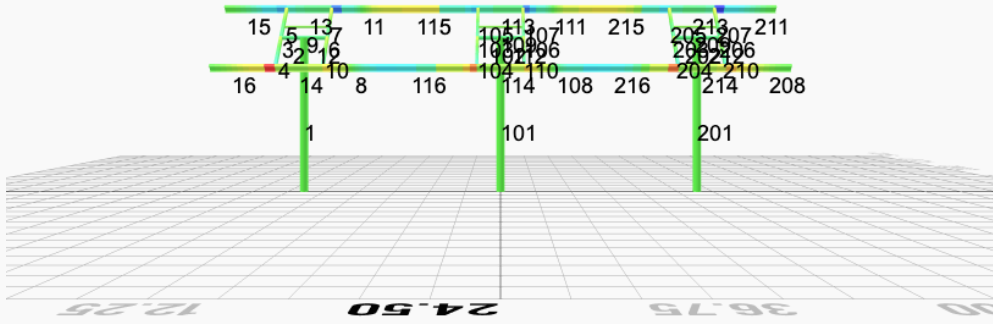
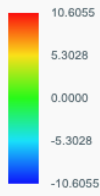
 SkyCiv



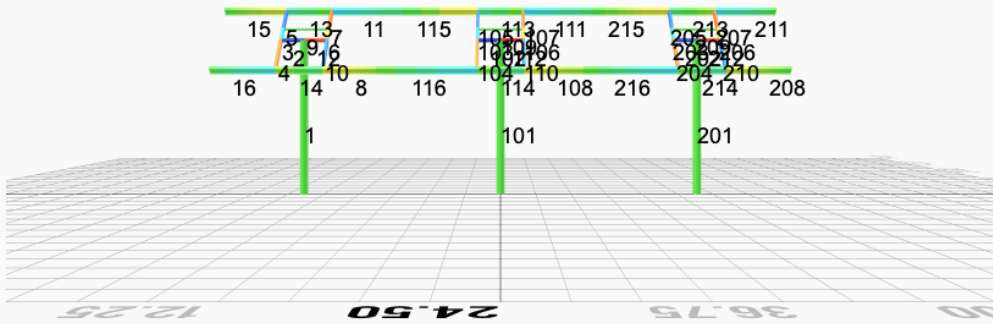
# FEM Results (Envelope Worst Case for each member)



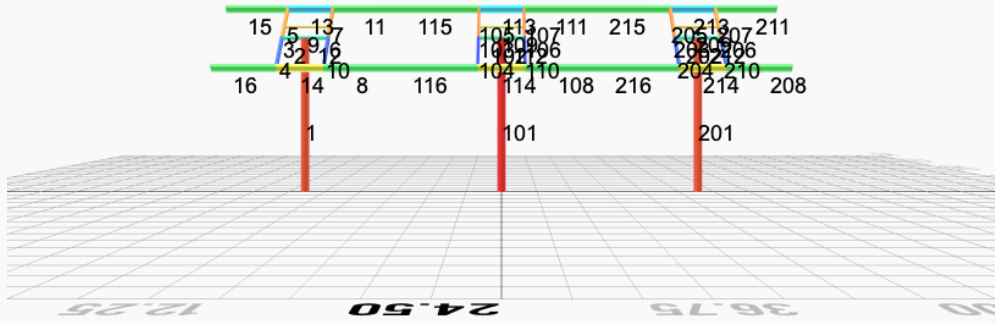
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



## 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	-0.0048	2.2134	-0.0032	-0.0110	0.0388	0.0791
ULS: 2. D + L	-0.0048	2.2134	-0.0032	-0.0110	0.0388	0.0791
ULS: 3. D + (S or Lr or R)	-0.0142	5.0313	-0.0093	-0.0317	0.1144	0.2011
ULS: 3. D + (S or Lr or R)	-0.0048	2.2134	-0.0032	-0.0110	0.0388	0.0791
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0118	4.3268	-0.0078	-0.0265	0.0955	0.1706
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0048	2.2134	-0.0032	-0.0110	0.0388	0.0791
ULS: 5b. D + 0.7E	-0.0048	2.2134	-0.0032	-0.0110	0.0388	0.0791
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0118	4.3268	-0.0078	-0.0265	0.0955	0.1706
ULS: 8. 0.6D + 0.7E	-0.0029	1.3281	-0.0019	-0.0066	0.0233	0.0474
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.0905	6.2903	-0.0140	-0.0548	0.0837	56.9811
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.0905	6.2903	-0.0140	-0.0548	0.0837	56.9811
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9494	-0.7350	0.0053	0.0230	0.0022	-37.8467
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6050	-0.3898	0.0028	0.0134	0.0156	-41.5079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0761	7.3844	-0.0159	-0.0594	0.1292	42.8472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.0761	7.3844	-0.0159	-0.0594	0.1292	42.8472
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2039	2.1155	-0.0014	-0.0010	0.0681	-28.2737
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9456	2.3744	-0.0033	-0.0083	0.0781	-31.0196
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0691	5.2711	-0.0113	-0.0438	0.0724	42.7556
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.0691	5.2711	-0.0113	-0.0438	0.0724	42.7556
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2109	0.0021	0.0032	0.0145	0.0114	-28.3653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9526	0.2610	0.0013	0.0073	0.0214	-31.1112
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.0886	5.4049	-0.0127	-0.0504	0.0682	56.9495
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.0886	5.4049	-0.0127	-0.0504	0.0682	56.9495
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9513	-1.6204	0.0066	0.0274	-0.0133	-37.8783
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6070	-1.2752	0.0041	0.0178	0.0001	-41.5395

### 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	10.8602
Shear X	-6.8207
Shear Z	-0.0247
Moment X	-0.0958
Moment Y (Twist)	0.2043
Moment Z	96.2022

### Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.3844
Shear X	-4.0905
Shear Z	-0.0159
Moment X	-0.0594
Moment Y (Twist)	0.1292
Moment Z	56.9811

## 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	0.0096	2.2829	-0.0001	-0.0006	0.0003	-0.0914
ULS: 2. D + L	0.0096	2.2829	-0.0001	-0.0006	0.0003	-0.0914
ULS: 3. D + (S or Lr or R)	0.0282	5.2390	-0.0002	-0.0018	0.0008	-0.3024
ULS: 3. D + (S or Lr or R)	0.0096	2.2829	-0.0001	-0.0006	0.0003	-0.0914
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0235	4.5000	-0.0002	-0.0015	0.0006	-0.2496

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0096	2.2829	-0.0001	-0.0006	0.0003	-0.0914
ULS: 5b. D + 0.7E	0.0096	2.2829	-0.0001	-0.0006	0.0003	-0.0914
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0235	4.5000	-0.0002	-0.0015	0.0006	-0.2496
ULS: 8. 0.6D + 0.7E	0.0057	1.3698	-0.0000	-0.0004	0.0002	-0.0548
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.1385	6.4487	-0.0006	-0.0045	0.0013	57.6230
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.1385	6.4487	-0.0006	-0.0045	0.0013	57.6230
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0131	-0.7321	0.0003	0.0022	-0.0005	-38.5740
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6534	-0.3741	0.0003	0.0019	-0.0004	-42.2227
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0876	7.6243	-0.0006	-0.0044	0.0014	43.0361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.0876	7.6243	-0.0006	-0.0044	0.0014	43.0361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2762	2.2387	0.0001	0.0006	0.0001	-29.1115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0064	2.5072	0.0001	0.0003	0.0002	-31.8481
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.1015	5.4073	-0.0005	-0.0035	0.0011	43.1944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.1015	5.4073	-0.0005	-0.0035	0.0011	43.1944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2622	0.0216	0.0002	0.0015	-0.0003	-28.9533
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9924	0.2901	0.0002	0.0012	-0.0002	-31.6898
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.1424	5.5356	-0.0006	-0.0043	0.0012	57.6595
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.1424	5.5356	-0.0006	-0.0043	0.0012	57.6595
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0093	-1.6453	0.0003	0.0024	-0.0006	-38.5374
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6496	-1.2873	0.0003	0.0021	-0.0005	-42.1861

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.1597
Shear X	-6.9135
Shear Z	-0.0011
Moment X	-0.0079
Moment Y (Twist)	0.0023
Moment Z	97.2053

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.6243
Shear X	-4.1424
Shear Z	-0.0006
Moment X	-0.0045
Moment Y (Twist)	0.0014
Moment Z	57.6595

### 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	-0.0048	2.2134	0.0033	0.0107	-0.0399	0.0784
ULS: 2. D + L	-0.0048	2.2134	0.0033	0.0107	-0.0399	0.0784
ULS: 3. D + (S or Lr or R)	-0.0140	5.0311	0.0095	0.0308	-0.1177	0.1992
ULS: 3. D + (S or Lr or R)	-0.0048	2.2134	0.0033	0.0107	-0.0399	0.0784
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0117	4.3267	0.0080	0.0257	-0.0983	0.1690
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0048	2.2134	0.0033	0.0107	-0.0399	0.0784
ULS: 5b. D + 0.7E	-0.0048	2.2134	0.0033	0.0107	-0.0399	0.0784
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0117	4.3267	0.0080	0.0257	-0.0983	0.1690
ULS: 8. 0.6D + 0.7E	-0.0029	1.3280	0.0020	0.0064	-0.0239	0.0471
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.0902	6.2899	0.0146	0.0528	-0.0906	56.9766
ULS: 5a. D + 0.6W_Wind downforce Case B only	-4.0902	6.2899	0.0146	0.0528	-0.0906	56.9766
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9493	-0.7349	-0.0056	-0.0222	0.0009	-37.8447
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.6049	-0.3897	-0.0030	-0.0127	-0.0129	-41.5064

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	-3.0758	7.3841	0.0165	0.0573	-0.1363	42.8426
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.0758	7.3841	0.0165	0.0573	-0.1363	42.8426
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2038	2.1155	0.0013	0.0010	-0.0677	-28.2733
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9455	2.3744	0.0032	0.0082	-0.0780	-31.0196
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0688	5.2708	0.0118	0.0423	-0.0779	42.7521
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-3.0688	5.2708	0.0118	0.0423	-0.0779	42.7521
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2108	0.0022	-0.0034	-0.0140	-0.0093	-28.3639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.9525	0.2611	-0.0015	-0.0068	-0.0196	-31.1102
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.0883	5.4045	0.0133	0.0485	-0.0746	56.9453
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-4.0883	5.4045	0.0133	0.0485	-0.0746	56.9453
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9512	-1.6202	-0.0069	-0.0265	0.0168	-37.8760
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.6068	-1.2750	-0.0044	-0.0169	0.0031	-41.5378

### 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	10.8595
Shear X	-6.8201
Shear Z	0.0258
Moment X	0.0922
Moment Y (Twist)	0.2139
Moment Z	96.1954

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.3841
Shear X	-4.0902
Shear Z	0.0165
Moment X	0.0573
Moment Y (Twist)	0.1363
Moment Z	56.9766

## Project Details

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

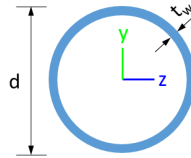


## Design Input Information

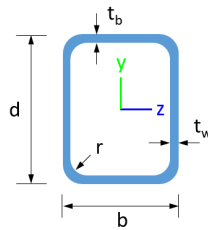
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

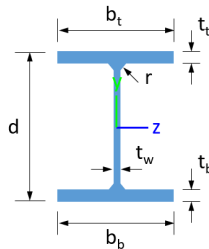
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



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



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

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
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101	574.32	212.13	123.94	123.94	172.30	172.30
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.57	6.12	40.24	43.62
114	133.20	85.85	23.71	6.12	40.24	43.62
115	133.20	58.22	25.58	6.12	40.24	43.62
116	133.20	86.20	24.49	6.12	40.24	43.62
201	574.32	212.13	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	52.83	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	52.83	32.87	6.12	40.24	43.62
212	198.33	182.14	21.95	21.95	59.50	59.50
213	133.20	85.85	25.56	6.12	40.24	43.62
214	133.20	85.85	24.38	6.12	40.24	43.62
215	133.20	58.22	26.42	6.12	40.24	43.62
216	133.20	58.22	26.58	6.12	40.24	43.62

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.051	0.776	0.002	0.040	0.000	0.803	#13	0.583	Not Required	Pass
2	0.002	0.395	0.301	0.086	0.058	0.696	#13	0.114	Not Required	Pass
3	0.010	0.754	0.064	0.075	0.013	0.791	#13	0.045	Not Required	Pass
4	0.010	0.678	0.132	0.068	0.028	0.725	#13	0.080	Not Required	Pass
5	0.010	0.468	0.137	0.075	0.033	0.483	#13	0.074	Not Required	Pass
6	0.010	0.745	0.068	0.074	0.014	0.782	#13	0.045	Not Required	Pass
7	0.010	0.463	0.134	0.074	0.034	0.480	#13	0.074	Not Required	Pass
8	0.000	0.063	0.091	0.043	0.015	0.141	#21	0.095	Not Required	Pass
9	0.008	0.055	0.058	0.001	0.000	0.115	#13	0.204	Not Required	Pass
10	0.010	0.667	0.131	0.067	0.029	0.720	#13	0.080	Not Required	Pass
11	0.000	0.070	0.090	0.047	0.015	0.142	#21	0.095	Not Required	Pass
12	0.002	0.384	0.294	0.085	0.057	0.678	#13	0.035	Not Required	Pass
13	0.006	0.243	0.338	0.064	0.020	0.511	#21	0.286	Not Required	Pass
14	0.008	0.226	0.338	0.058	0.020	0.501	#21	0.190	Not Required	Pass
15	0.000	0.082	0.157	0.036	0.012	0.220	#21	Not Required	Not Required	Pass

16	0.000	0.074	0.157	0.032	0.012	0.216	#21	Not Required	Not Required	Pass
101	0.053	0.784	0.000	0.040	0.000	0.811	#13	0.583	Not Required	Pass
102	0.001	0.396	0.297	0.088	0.058	0.693	#13	0.035	Not Required	Pass
103	0.010	0.765	0.074	0.076	0.018	0.806	#13	0.045	Not Required	Pass
104	0.010	0.689	0.123	0.069	0.027	0.740	#13	0.080	Not Required	Pass
105	0.010	0.475	0.126	0.076	0.031	0.490	#13	0.074	Not Required	Pass
106	0.010	0.764	0.074	0.076	0.018	0.806	#13	0.045	Not Required	Pass
107	0.010	0.475	0.126	0.076	0.031	0.490	#13	0.074	Not Required	Pass
108	0.000	0.075	0.092	0.039	0.015	0.149	#21	0.095	Not Required	Pass
109	0.005	0.048	0.053	0.001	0.000	0.103	#13	0.204	Not Required	Pass
110	0.010	0.689	0.123	0.069	0.027	0.740	#13	0.080	Not Required	Pass
111	0.000	0.084	0.092	0.043	0.015	0.151	#21	0.095	Not Required	Pass
112	0.001	0.396	0.297	0.088	0.058	0.693	#13	0.035	Not Required	Pass
113	0.005	0.171	0.301	0.060	0.020	0.421	#21	0.286	Not Required	Pass
114	0.007	0.165	0.300	0.054	0.020	0.412	#21	0.286	Not Required	Pass
115	0.000	0.149	0.170	0.043	0.015	0.282	#21	0.532	Not Required	Pass
116	0.000	0.134	0.171	0.039	0.015	0.276	#21	0.346	Not Required	Pass
201	0.051	0.776	0.002	0.040	0.000	0.803	#13	0.583	Not Required	Pass
202	0.002	0.384	0.293	0.085	0.057	0.677	#13	0.035	Not Required	Pass
203	0.010	0.745	0.068	0.074	0.014	0.782	#13	0.045	Not Required	Pass
204	0.010	0.667	0.131	0.067	0.029	0.719	#13	0.080	Not Required	Pass
205	0.010	0.463	0.134	0.074	0.034	0.480	#13	0.074	Not Required	Pass
206	0.010	0.755	0.064	0.075	0.013	0.792	#13	0.045	Not Required	Pass
207	0.010	0.468	0.137	0.075	0.033	0.483	#13	0.074	Not Required	Pass
208	0.000	0.074	0.157	0.032	0.012	0.216	#21	Not Required	Not Required	Pass
209	0.008	0.055	0.058	0.001	0.000	0.115	#13	0.204	Not Required	Pass
210	0.010	0.679	0.132	0.068	0.028	0.725	#13	0.080	Not Required	Pass
211	0.000	0.082	0.157	0.036	0.012	0.220	#21	Not Required	Not Required	Pass
212	0.002	0.395	0.301	0.086	0.058	0.696	#13	0.114	Not Required	Pass
213	0.006	0.243	0.338	0.064	0.020	0.511	#21	0.190	Not Required	Pass
214	0.008	0.226	0.338	0.058	0.020	0.500	#21	0.286	Not Required	Pass
215	0.000	0.145	0.170	0.047	0.015	0.279	#21	0.532	Not Required	Pass
216	0.000	0.131	0.171	0.043	0.015	0.274	#21	0.532	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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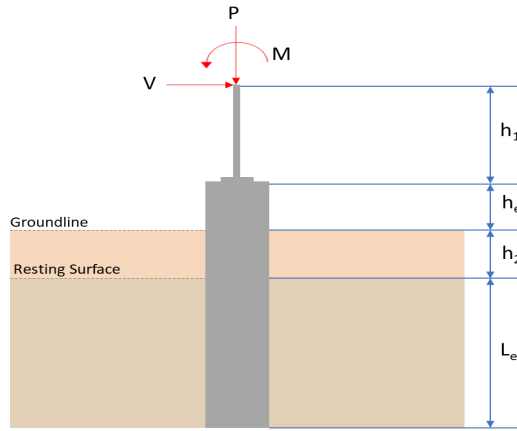
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 8.25$  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)	7.384	10.860
$V_x$ (kip)	-4.091	-6.821
$V_z$ (kip)	-0.016	-0.025
$M_x$ (kipft)	-0.059	-0.096
$M_z$ (kipft)	56.981	96.202

### 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,

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

$$H_o = \frac{(-4.091 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.65143 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(56.981 \text{ kipft}) + ((-4.091 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.0734 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5519 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.016 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0025478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.059 \text{ kipft}) + ((-0.016 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.009395 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.85302 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.5519 \text{ ft}), (0.85302 \text{ ft})]$$

$$L_{e,req} = 7.552 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.552 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91539$$

Status: **PASS**  
Ratio: **0.920**

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(7.384 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.4615 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4615 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23075$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.65143 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 9.0734 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (9.0734 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.65143 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.0734 \text{ kipft/ft})) + (4 \times (-0.65143 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6946 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.0734 \text{ kipft/ft})) + (3 \times (-0.65143 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.0734 \text{ kipft/ft})) + (2 \times (-0.65143 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.27218 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (9.0734 \text{ kipft/ft})) + ((-0.65143 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1259 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6946 \text{ ft})}{2}$$

$$p_a = 0.4271 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.27218 \text{ kip/ft}^2)}{(0.4271 \text{ kip/ft}^2)}$$

$$Ratio = 0.63728$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1259 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.90986$$

Status: **PASS**  
Ratio: **0.640**

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

#### Considering z-direction:

$H_o = -0.0025478 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.009395 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.009395 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.009395 \text{ kipft/ft})) + (4 \times (-0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9116 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.009395 \text{ kipft/ft})) + (3 \times (-0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.009395 \text{ kipft/ft})) + (2 \times (-0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = -0.00051631 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.009395 \text{ kipft/ft})) + ((-0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = -0.00019652 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9116 \text{ ft})}{2}$$

$$p_a = 0.44337 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00051631 \text{ kip/ft}^2)}{(0.44337 \text{ kip/ft}^2)}$$

$$Ratio = -0.0011645$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

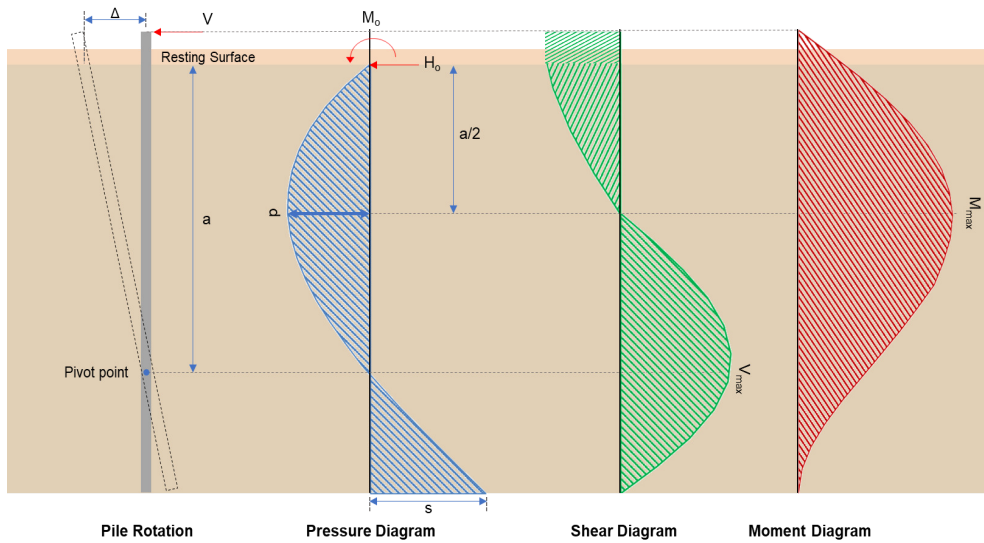
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00019652 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = -0.00015881$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-6.821 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0861 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(96.202 \text{ kipft}) + ((-6.821 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.319 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.319 \text{ kipft/ft})}{(-1.0861 \text{ kip/ft})}$$

$$E = 14.104 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.319 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.0861 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 15.319) + (4 \times (-1.0861) \times 8.25)}$$

$$a = \frac{(6 \times (15.319 \text{ kipft/ft})) + (4 \times (-1.0861 \text{ kip/ft}) \times (8.25 \text{ ft}))}{(6 \times (15.319 \text{ kipft/ft})) + (4 \times (-1.0861 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6929 \text{ ft}$$

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

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

$$V_{max} = ((-1.0861 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (14.104 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6929 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (14.104 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6929 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.008 \text{ kip}$$

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

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

$$M_{max} = ((-1.0861 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(14.104 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (14.104 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (14.104 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 62.779 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.025 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0039809 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.096 \text{ kipft}) + ((-0.025 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015287 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.015287 \text{ kipft/ft})}{(-0.0039809 \text{ kip/ft})}$$

$$E = 3.84 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.015287 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.0039809 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.015287 \text{ kipft/ft})) + (4 \times (-0.0039809 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9048 \text{ ft}$$

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

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

$$V_{max} = ((-0.0039809 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.84 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.9048 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.84 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.9048 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023736 \text{ kip}$$

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

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

$$M_{max} = ((-0.0039809 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(3.84 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.9048 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.84 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.9048 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.84 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.9048 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.086205 \text{ kipft}$$

### 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,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.86 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.235 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.235 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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 = 0.96556</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><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</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 0.80 [(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 0.80 \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))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \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{(10.86 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0040595</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 = 48 \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 (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \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{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</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 (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.86 \text{ kip} \rightarrow 10860 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10860 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.93 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.93 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.93 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.04 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 16.008 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.008 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.14417$$

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

**Considering z-direction:**

$V_{max} = 0.023736 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.023736 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.00021377$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 62.779 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(62.779 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.25152$$

Status: **PASS**  
Ratio: **0.250**

**Considering z-direction:**

$M_{max} = 0.086205 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.086205 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00034537$$

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

REFERENCES	CALCULATIONS	RESULTS
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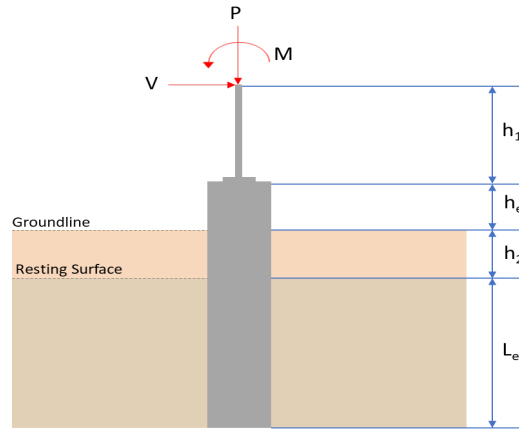
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 8.25$  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)	7.624	11.160
$V_x$ (kip)	-4.142	-6.913
$V_z$ (kip)	-0.001	-0.001
$M_x$ (kipft)	-0.005	-0.008
$M_z$ (kipft)	57.660	97.205

### 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,

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

$$H_o = \frac{(-4.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.65955 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(57.66 \text{ kipft}) + ((-4.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.1815 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5755 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.001 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00015924 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.005 \text{ kipft}) + ((-0.001 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00079618 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.392 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.5755 \text{ ft}), (0.392 \text{ ft})]$$

$$L_{e,req} = 7.575 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.575 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91818$$

Status: **PASS**  
Ratio: **0.920**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(7.624 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.4765 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4765 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23825$$

Status: **PASS**  
Ratio: **0.240**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.65955 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 9.1815 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (9.1815 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.65955 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.1815 \text{ kipft/ft})) + (4 \times (-0.65955 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6947 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.1815 \text{ kipft/ft})) + (3 \times (-0.65955 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.1815 \text{ kipft/ft})) + (2 \times (-0.65955 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.27528 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (9.1815 \text{ kipft/ft})) + ((-0.65955 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1391 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6947 \text{ ft})}{2}$$

$$p_a = 0.4271 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27528 \text{ kip/ft}^2)}{(0.4271 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64454$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1391 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92049$$

Status: **PASS**  
Ratio: **0.640**

Status: **PASS**  
Ratio: **0.920**

#### Considering z-direction:

$H_o = -0.00015924 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.00079618 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.00079618 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.00079618 \text{ kipft/ft})) + (4 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.8601 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.00079618 \text{ kipft/ft})) + (3 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.00079618 \text{ kipft/ft})) + (2 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = -0.000026393 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.00079618 \text{ kipft/ft})) + ((-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.000024565 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8601 \text{ ft})}{2}$$

$$p_a = 0.43951 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.000026393 \text{ kip/ft}^2)}{(0.43951 \text{ kip/ft}^2)}$$

$$Ratio = -0.000060051$$

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

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

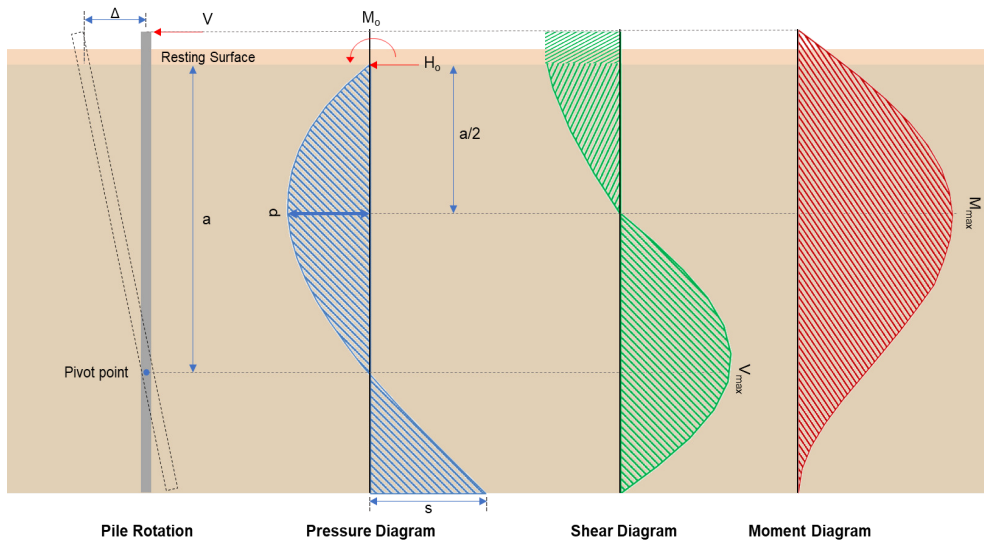
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000024565 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.000019851$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-6.913 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1008 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(97.205 \text{ kipft}) + ((-6.913 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.479 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.479 \text{ kipft/ft})}{(-1.1008 \text{ kip/ft})}$$

$$E = 14.061 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.479 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.1008 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 15.479 \text{ kipft/ft}) + (4 \times (-1.1008 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (15.479 \text{ kipft/ft})) + (4 \times (-1.1008 \text{ kip/ft}) \times (8.25 \text{ ft}))}{}$$

$$a = 5.6933 \text{ ft}$$

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

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

$$V_{max} = ((-1.1008 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (14.061 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6933 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (14.061 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6933 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.184 \text{ kip}$$

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

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

$$M_{max} = ((-1.1008 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(14.061 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6933 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (14.061 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6933 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (14.061 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6933 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 63.46 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.001 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00015924 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.008 \text{ kipft}) + ((-0.001 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0012739 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.0012739 \text{ kipft/ft})}{(-0.00015924 \text{ kip/ft})}$$

$$E = 8 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.0012739 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.0012739 \text{ kipft/ft})) + (4 \times (-0.00015924 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7801 \text{ ft}$$

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

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

$$V_{max} = ((-0.00015924 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.7801 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.7801 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0015137 \text{ kip}$$

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

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

$$M_{max} = ((-0.00015924 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(8 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.7801 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.7801 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.7801 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.0057709 \text{ kipft}$$

### 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,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(11.16 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.225 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.225 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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 = 0.96556</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><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</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 0.80 [(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 0.80 \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))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \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{(11.16 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0041717</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 = 48 \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 (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \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{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</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 (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 11.16 \text{ kip} \rightarrow 11160 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11160 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.97 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.97 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.97 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.97 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.06 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 16.184 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.184 \text{ kip})}{(111.06 \text{ kip})}$$

$$Ratio = 0.14572$$

Status: **PASS**  
Ratio: **0.150**

**Considering z-direction:**

$V_{max} = 0.0015137 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.0015137 \text{ kip})}{(111.06 \text{ kip})}$$

$$Ratio = 0.000013629$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 63.46 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(63.46 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.25425$$

Status: **PASS**  
Ratio: **0.250**

**Considering z-direction:**

$M_{max} = 0.0057709 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.0057709 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.000023121$$

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

REFERENCES	CALCULATIONS	RESULTS
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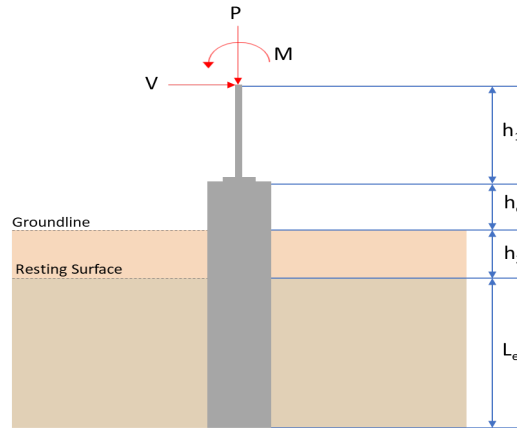
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 8.25$  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)	7.384	10.860
$V_x$ (kip)	-4.090	-6.820
$V_z$ (kip)	0.016	0.026
$M_x$ (kipft)	0.057	0.092
$M_z$ (kipft)	56.977	96.195

### 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,

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

$$H_o = \frac{(-4.09 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.65127 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(56.977 \text{ kipft}) + ((-4.09 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.0728 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.552 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.016 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0025478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.057 \text{ kipft}) + ((0.016 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0090764 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.95559 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.552 \text{ ft}), (0.95559 \text{ ft})]$$

$$L_{e,req} = 7.552 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.552 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91539$$

Status: **PASS**  
Ratio: **0.920**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(7.384 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.4615 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4615 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23075$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.65127 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 9.0728 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (9.0728 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.65127 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (9.0728 \text{ kipft/ft})) + (4 \times (-0.65127 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6946 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.0728 \text{ kipft/ft})) + (3 \times (-0.65127 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (9.0728 \text{ kipft/ft})) + (2 \times (-0.65127 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.27221 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (9.0728 \text{ kipft/ft})) + ((-0.65127 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.126 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6946 \text{ ft})}{2}$$

$$p_a = 0.4271 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27221 \text{ kip/ft}^2)}{(0.4271 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63735$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.126 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90986$$

Status: **PASS**  
Ratio: **0.640**

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

#### Considering z-direction:

$H_o = 0.0025478 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.0090764 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.0090764 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.0090764 \text{ kipft/ft})) + (4 \times (0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9172 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0090764 \text{ kipft/ft})) + (3 \times (0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (0.0090764 \text{ kipft/ft})) + (2 \times (0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.0015706 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.0090764 \text{ kipft/ft})) + ((0.0025478 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 0.0034532 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9172 \text{ ft})}{2}$$

$$p_a = 0.44379 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0015706 \text{ kip/ft}^2)}{(0.44379 \text{ kip/ft}^2)}$$

$$Ratio = 0.0035391$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

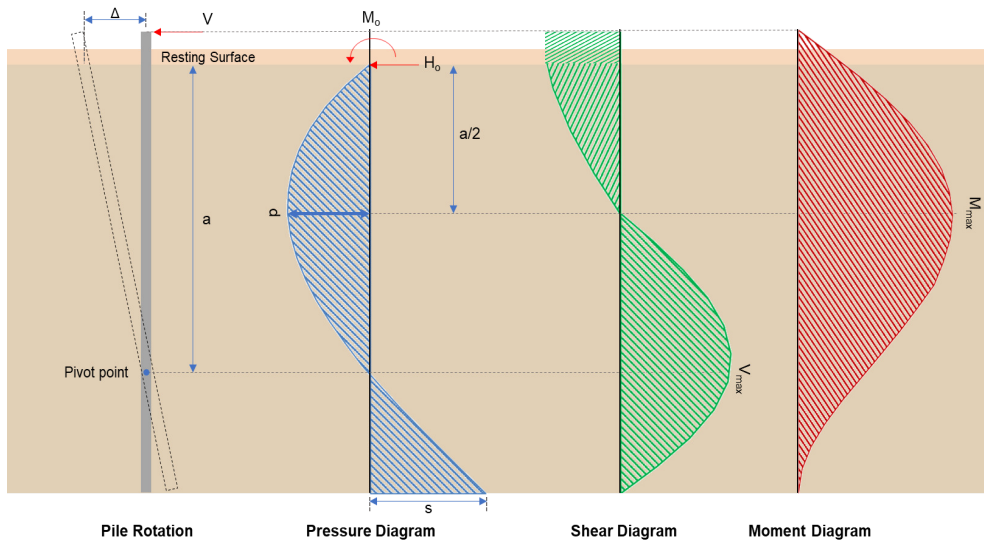
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0034532 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0027904$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-6.82 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.086 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(96.195 \text{ kipft}) + ((-6.82 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.318 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.318 \text{ kipft/ft})}{(-1.086 \text{ kip/ft})}$$

$$E = 14.105 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.318 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.086 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times 15.318 \text{ kipft/ft}) + (4 \times (-1.086 \text{ kip/ft}) \times 8.25 \text{ ft})}$$

$$a = \frac{(6 \times (15.318 \text{ kipft/ft})) + (4 \times (-1.086 \text{ kip/ft}) \times (8.25 \text{ ft}))}{(6 \times (15.318 \text{ kipft/ft})) + (4 \times (-1.086 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6929 \text{ ft}$$

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

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

$$V_{max} = ((-1.086 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (14.105 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6929 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (14.105 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6929 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.007 \text{ kip}$$

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

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

$$M_{max} = ((-1.086 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(14.105 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (14.105 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (14.105 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.6929 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 62.774 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0041401 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.092 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.01465 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.01465 \text{ kipft/ft})}{(0.0041401 \text{ kip/ft})}$$

$$E = 3.5385 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.01465 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0.0041401 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.01465 \text{ kipft/ft})) + (4 \times (0.0041401 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.9184 \text{ ft}$$

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

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

$$V_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.5385 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.9184 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.5385 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.9184 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023628 \text{ kip}$$

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

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

$$M_{max} = ((0.0041401 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[ \left( \frac{(3.5385 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.9184 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.5385 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.9184 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.5385 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.9184 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.085306 \text{ kipft}$$

### 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,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.86 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.235 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.235 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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 = 0.96556</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><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</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 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]</math></p> <p style="text-align: center;"><math>\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))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \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{(10.86 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0040595</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 = 48 \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 (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \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{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</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 (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.86 \text{ kip} \rightarrow 10860 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10860 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.93 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.93 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.93 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.04 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 16.007 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.007 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.14416$$

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

**Considering z-direction:**

$V_{max} = 0.023628 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.023628 \text{ kip})}{(111.04 \text{ kip})}$$

$$Ratio = 0.0002128$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 62.774 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(62.774 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2515$$

Status: **PASS**  
Ratio: **0.250**

**Considering z-direction:**

$M_{max} = 0.085306 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.085306 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00034177$$

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