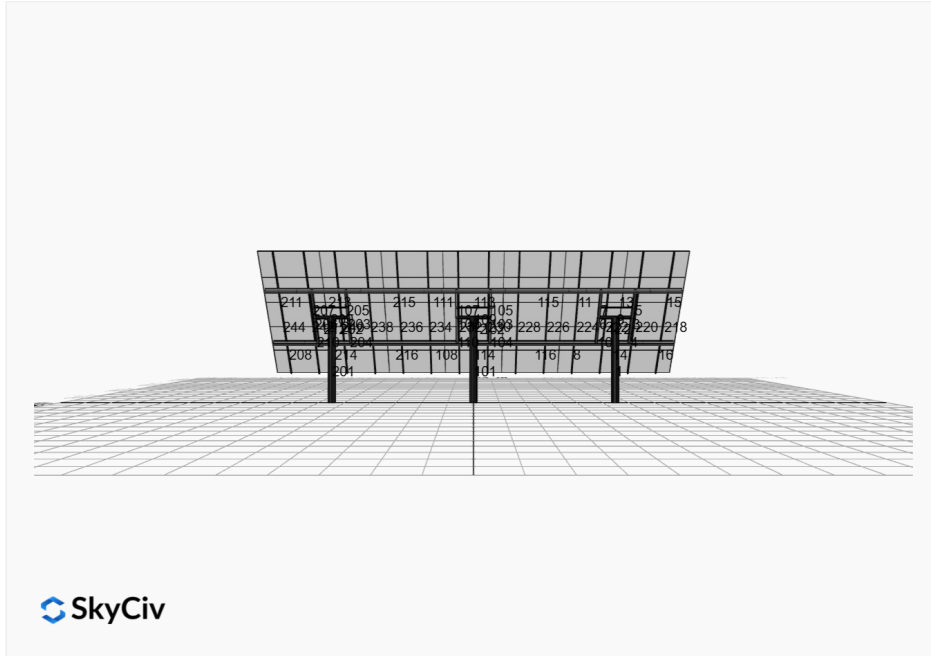


**Project Name:** test 8 **Date:** Mon Nov 04 2024  
**Location:** 28385 Post Office Rd, Mass City, MI 49948, **Number of Modules:** 35  
 USA **Number of Poles:** 3  
**Unique ID:** 3P-17-10TOP-HD-45-L-5Hx7W-8C1D **Date Sold:**  
**Dealer:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	17.42 ft
<b>Array Dimensions E/W</b>	49.53 ft
<b>Winter Tilt Angle</b>	55.8
<b>Front Edge Clearance</b>	3 ft

### MT Solar Bill of Materials (3P-17-10TOP-HD-45-L-5Hx7W-8C1D)

Part	Short Description	BOM Qty
MTS-PC-10	10IN 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-ANGLE-4PK	Angle Clamp	7

### Rail Bill of Materials

Part	Qty
Rails (207in)	14
Rail Attachment	56
Module Mid Clamp	56
Module End Clamp	28
Ground Lug	7

## Site Details:



**Site Address:** 28385 Post Office Rd, Mass City, MI 49948, USA

### Array Specification

<b>Duty Classification:</b>	HD
<b>Module Width:</b>	41.30 in
<b>Module Length:</b>	83.90in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	7
<b>Total Number of Modules:</b>	35
<b>Winter Tilt Angle:</b>	55.8
<b>Front Edge Clearance:</b>	3
<b>Total Array Height at Tilt:</b>	17.40 ft
<b>Total Frame Length:</b>	49.00 ft
<b>Frame Weight:</b>	3549 lbs
<b>Array Dimensions N/S:</b>	17.42 ft
<b>Array Dimensions E/W:</b>	49.53 ft
<b>Rail Length:</b>	209.00 in
<b>Rail Spacing:</b>	3.54 ft

### Support Specifications

<b>Pole Size:</b>	10in Pipe Sch 40
<b>Pole Length above Grade:</b>	10.20 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: 7.25 ft Pile 2: 7.25 ft Pile 3: 7.25 ft
<b>Foundation Volume:</b>	12.889 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	28385 Post Office Rd, Mass City, MI 49948, USA
<b>Wind Speed:</b>	120 mph
<b>Snow Load:</b>	80 psf

### **Design Disclaimer**

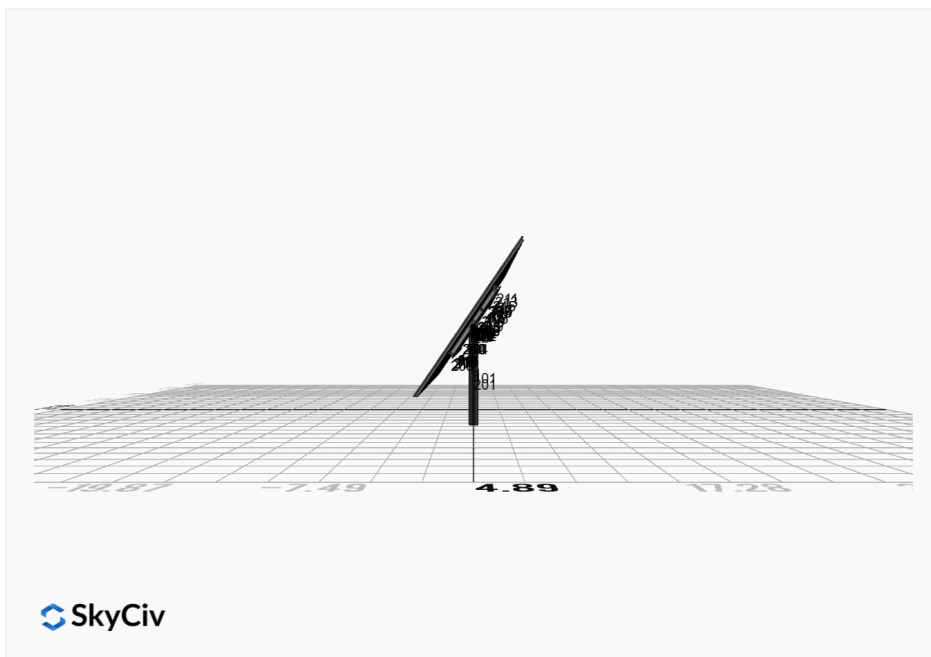
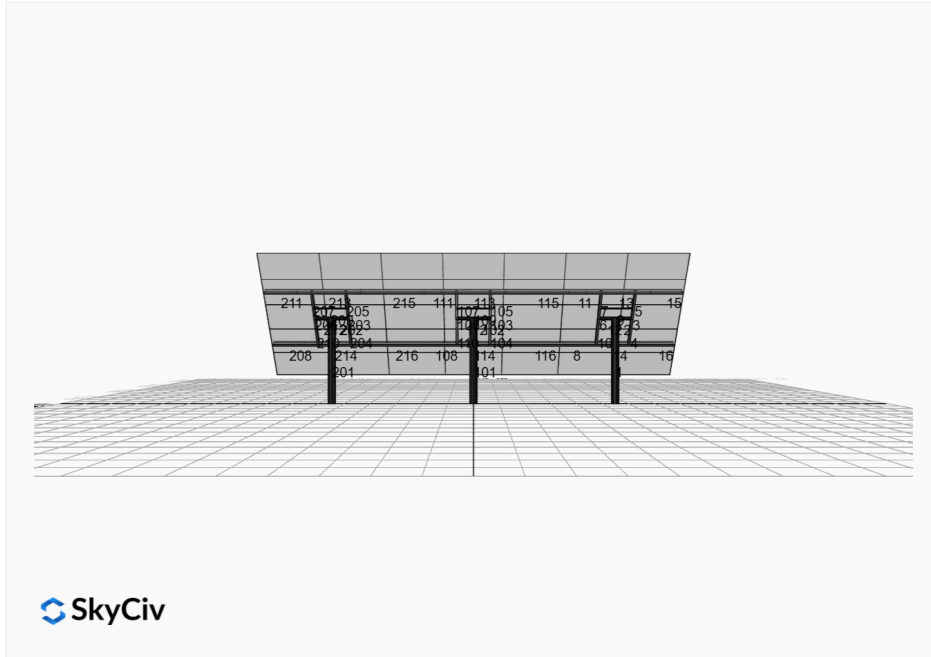
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

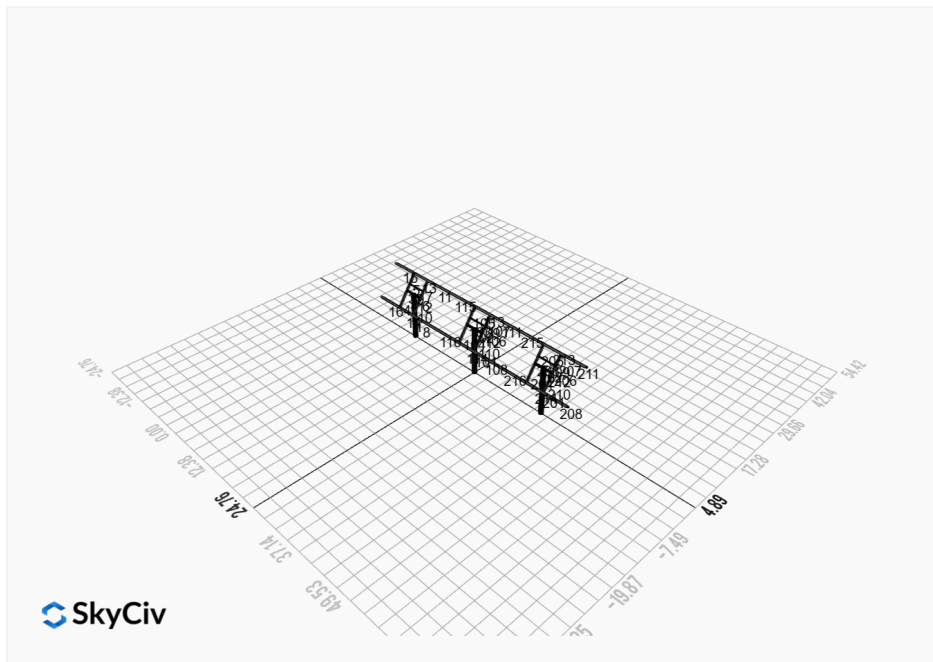
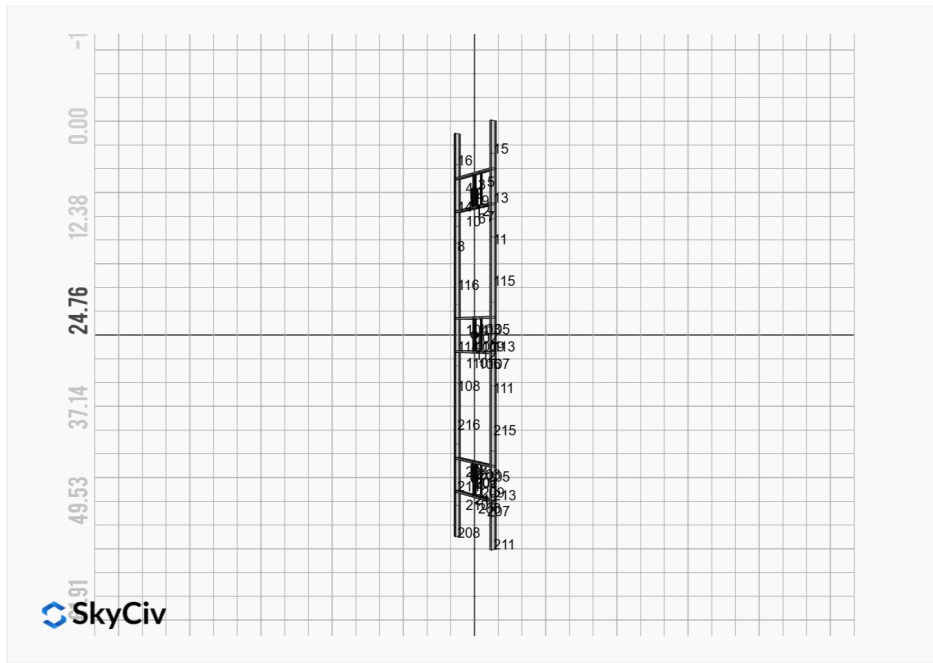
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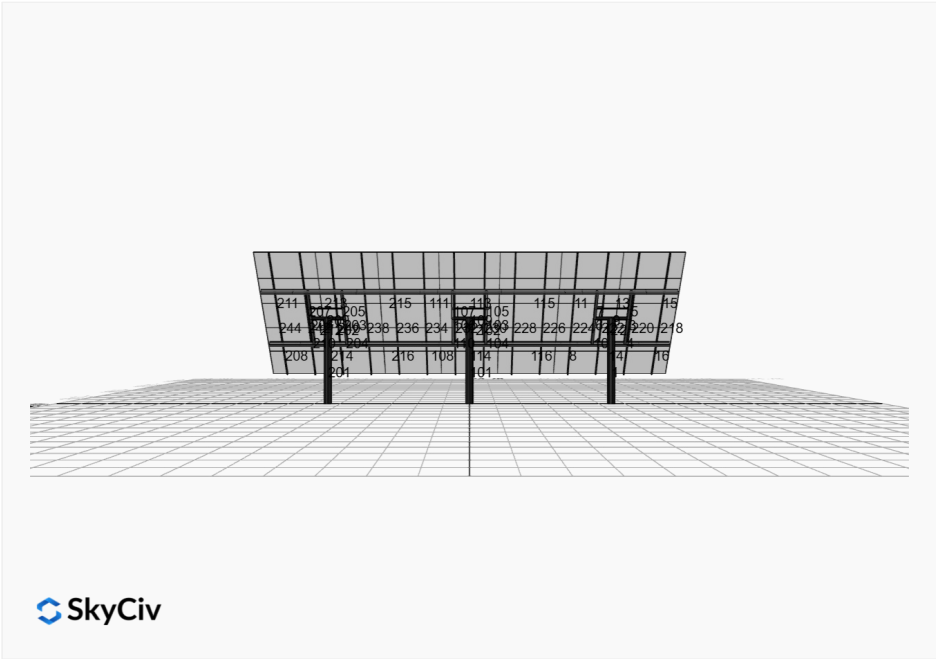
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## Design Notes:

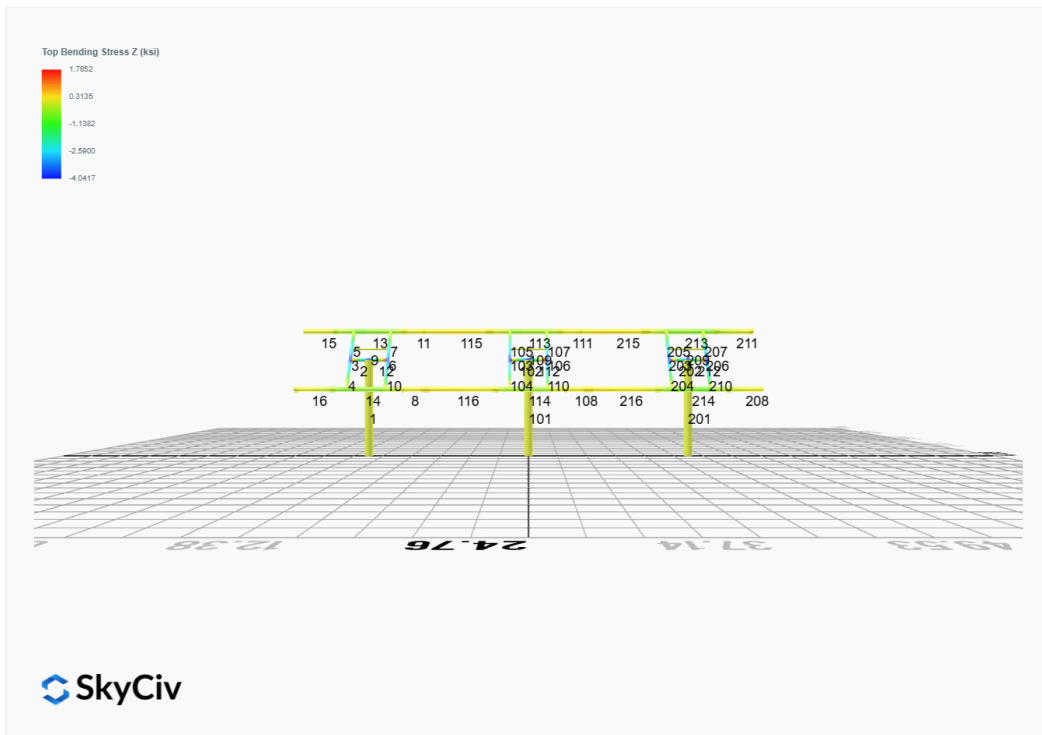
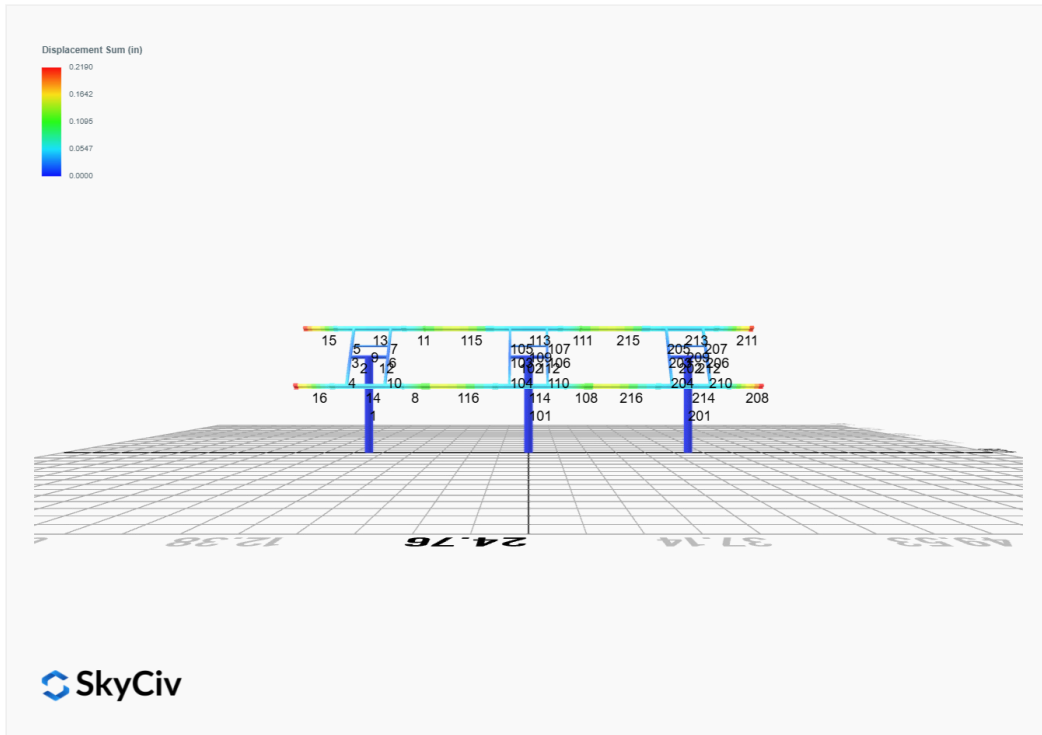
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

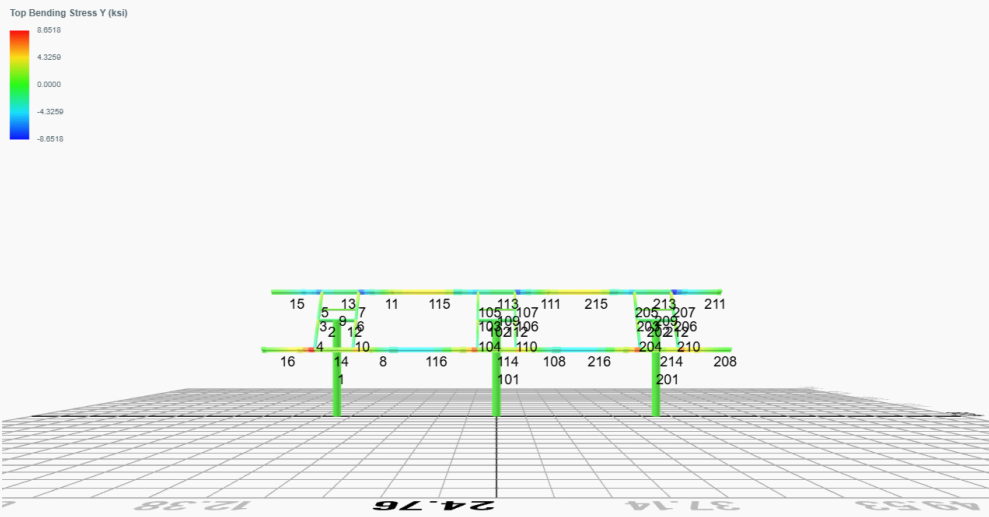




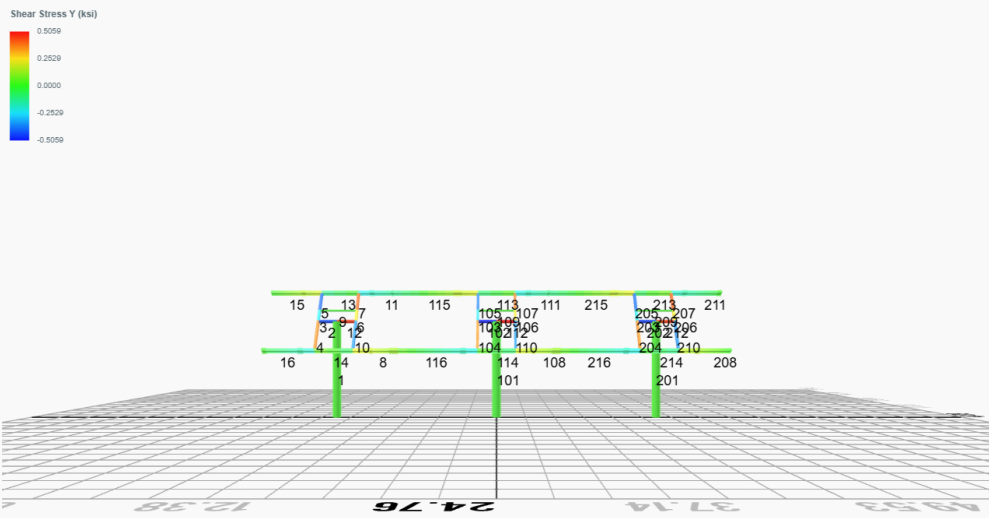


# FEM Results (Envelope Worst Case for each member)

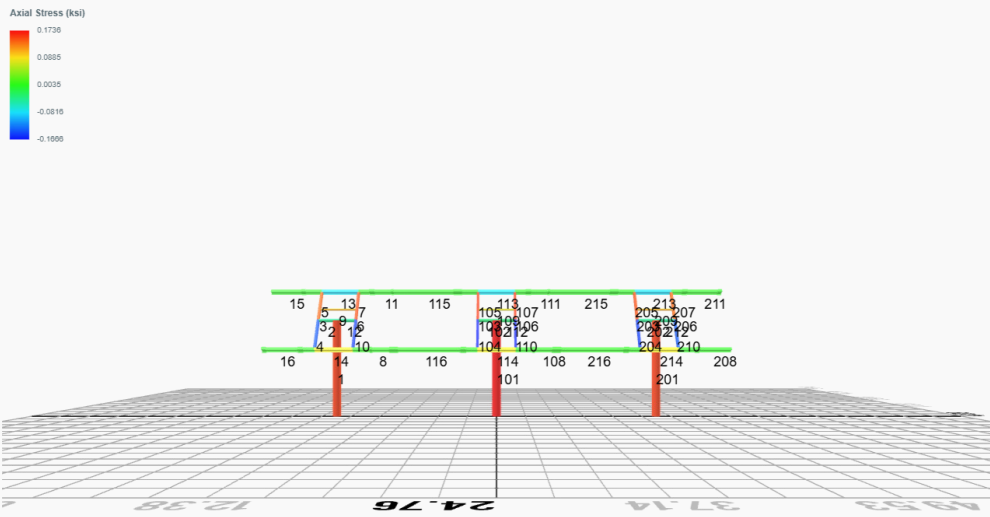




SkyCiv



SkyCiv



## 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	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 2. D + L	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 3. D + (S or Lr or R)	-0.0151	4.1788	0.0029	0.0152	0.1183	0.1670
ULS: 3. D + (S or Lr or R)	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0130	3.6881	0.0025	0.0130	0.1020	0.1460
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 5b. D + 0.7E	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0130	3.6881	0.0025	0.0130	0.1020	0.1460
ULS: 8. 0.6D + 0.7E	-0.0041	1.3297	0.0008	0.0039	0.0320	0.0499
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.4314	5.2203	-0.0115	-0.0175	0.1562	45.4862
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4176	-0.7879	0.0140	0.0306	-0.0504	-44.8325
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3314	5.9413	-0.0070	-0.0050	0.1792	34.1983
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0130	3.6881	0.0025	0.0130	0.1020	0.1460
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3053	1.4351	0.0121	0.0311	0.0242	-33.5407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0130	3.6881	0.0025	0.0130	0.1020	0.1460
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3252	4.4693	-0.0083	-0.0115	0.1305	34.1354
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3115	-0.0369	0.0108	0.0246	-0.0245	-33.6036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0068	2.2162	0.0013	0.0065	0.0533	0.0831
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.4287	4.3339	-0.0120	-0.0201	0.1349	45.4529
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0041	1.3297	0.0008	0.0039	0.0320	0.0499
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4203	-1.6744	0.0135	0.0280	-0.0717	-44.8658
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0041	1.3297	0.0008	0.0039	0.0320	0.0499

### 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	8.6477
Shear X	-7.3868
Shear Z	0.0236
Moment X	0.0521
Moment Y (Twist)	0.2675
Moment Z	76.1791

### 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	5.9413
Shear X	-4.4314
Shear Z	0.0140
Moment X	0.0311
Moment Y (Twist)	0.1792
Moment Z	45.4862

## 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	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 2. D + L	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 3. D + (S or Lr or R)	0.0301	4.3676	-0.0000	0.0000	0.0000	-0.2652
ULS: 3. D + (S or Lr or R)	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0260	3.8508	-0.0000	0.0000	0.0000	-0.2268

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 5b. D + 0.7E	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0260	3.8508	-0.0000	0.0000	0.0000	-0.2268
ULS: 8. 0.6D + 0.7E	0.0082	1.3804	-0.0000	0.0000	0.0000	-0.0670
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5071	5.3784	-0.0000	0.0000	0.0000	46.2314
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.5348	-0.7774	-0.0000	0.0000	0.0000	-45.9531
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3645	6.1592	-0.0000	0.0000	0.0000	34.5305
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0260	3.8508	-0.0000	0.0000	0.0000	-0.2268
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4169	1.5423	-0.0000	0.0000	0.0000	-34.6079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0260	3.8508	-0.0000	0.0000	0.0000	-0.2268
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3770	4.6090	-0.0000	0.0000	0.0000	34.6457
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4045	-0.0079	-0.0000	0.0000	0.0000	-34.4928
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0136	2.3006	-0.0000	0.0000	0.0000	-0.1117
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5126	4.4582	-0.0000	0.0000	0.0000	46.2761
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0082	1.3804	-0.0000	0.0000	0.0000	-0.0670
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.5293	-1.6977	-0.0000	0.0000	0.0000	-45.9085
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0082	1.3804	-0.0000	0.0000	0.0000	-0.0670

### 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	8.9238
Shear X	-7.5595
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0002
Moment Z	77.4063

### 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	6.1592
Shear X	-4.5348
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	46.2761

### 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	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 2. D + L	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 3. D + (S or Lr or R)	-0.0151	4.1788	-0.0029	-0.0152	-0.1183	0.1670
ULS: 3. D + (S or Lr or R)	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0130	3.6881	-0.0025	-0.0130	-0.1020	0.1460
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 5b. D + 0.7E	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0130	3.6881	-0.0025	-0.0130	-0.1020	0.1460
ULS: 8. 0.6D + 0.7E	-0.0041	1.3297	-0.0008	-0.0039	-0.0320	0.0499
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.4314	5.2203	0.0115	0.0175	-0.1562	45.4862
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.4176	-0.7879	-0.0140	-0.0306	0.0504	-44.8325
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831

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	-3.3314	5.9413	0.0070	0.0050	-0.1792	34.1983
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0130	3.6881	-0.0025	-0.0130	-0.1020	0.1460
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3053	1.4351	-0.0121	-0.0311	-0.0242	-33.5407
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0130	3.6881	-0.0025	-0.0130	-0.1020	0.1460
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3252	4.4693	0.0083	0.0115	-0.1305	34.1354
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3115	-0.0369	-0.0108	-0.0246	0.0245	-33.6036
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0068	2.2162	-0.0013	-0.0065	-0.0533	0.0831
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.4287	4.3339	0.0120	0.0201	-0.1349	45.4529
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0041	1.3297	-0.0008	-0.0039	-0.0320	0.0499
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4203	-1.6744	-0.0135	-0.0280	0.0718	-44.8658
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0041	1.3297	-0.0008	-0.0039	-0.0320	0.0499

### 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	8.6478
Shear X	-7.3868
Shear Z	-0.0236
Moment X	-0.0523
Moment Y (Twist)	0.2672
Moment Z	76.1799

### 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	5.9413
Shear X	-4.4314
Shear Z	-0.0140
Moment X	-0.0311
Moment Y (Twist)	0.1792
Moment Z	45.4862

# Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States  
  
 User Name: sales@mtsolar.us  
 Project Name: test 8  
 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					
11	10in Pipe Sch 40	10.75	0.36					

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> )	$I_{yD}$ (in <sup>4</sup> )	$I_{zD}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yD}$ (in <sup>3</sup> )	$S_{zD}$ (in <sup>3</sup> )





101	535.87	374.59	147.68	147.68	160.76	160.76
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	24.23	6.12	40.24	43.62
114	133.20	85.85	24.24	6.12	40.24	43.62
115	133.20	86.20	25.22	6.12	40.24	43.62
116	133.20	86.20	25.64	6.12	40.24	43.62
201	535.87	374.59	147.68	147.68	160.76	160.76
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	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	25.52	6.12	40.24	43.62
214	133.20	85.85	25.51	6.12	40.24	43.62
215	133.20	86.20	26.65	6.12	40.24	43.62
216	133.20	86.20	26.68	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	0.023	0.516	0.001	0.046	0.000	0.528	#13	0.350	Not Required	Pass
2	0.002	0.306	0.323	0.069	0.063	0.629	#13	0.035	Not Required	Pass
3	0.009	0.639	0.062	0.064	0.013	0.679	#13	0.045	Not Required	Pass
4	0.009	0.636	0.128	0.064	0.027	0.688	#13	0.080	Not Required	Pass
5	0.009	0.396	0.130	0.064	0.032	0.413	#13	0.074	Not Required	Pass
6	0.010	0.627	0.065	0.063	0.013	0.668	#13	0.045	Not Required	Pass
7	0.010	0.390	0.126	0.062	0.032	0.408	#13	0.074	Not Required	Pass
8	0.000	0.060	0.087	0.040	0.014	0.128	#21	0.095	Not Required	Pass
9	0.007	0.038	0.059	0.001	0.000	0.099	#13	0.204	Not Required	Pass
10	0.009	0.623	0.122	0.063	0.027	0.680	#13	0.080	Not Required	Pass
11	0.000	0.059	0.085	0.040	0.014	0.126	#21	0.095	Not Required	Pass
12	0.002	0.298	0.311	0.068	0.062	0.609	#13	0.035	Not Required	Pass
13	0.006	0.207	0.322	0.054	0.019	0.455	#21	0.286	Not Required	Pass
14	0.007	0.211	0.322	0.054	0.019	0.455	#21	0.190	Not Required	Pass
15	0.000	0.070	0.140	0.020	0.011	0.100	#21	Not Required	Not Required	Pass

15	0.000	0.070	0.149	0.030	0.011	0.198	#21	Not Required	Not Required	Pass
16	0.000	0.070	0.149	0.030	0.011	0.198	#21	Not Required	Not Required	Pass
101	0.024	0.524	0.000	0.047	0.000	0.536	#13	0.350	Not Required	Pass
102	0.001	0.309	0.319	0.071	0.064	0.628	#13	0.035	Not Required	Pass
103	0.010	0.647	0.071	0.065	0.017	0.693	#13	0.045	Not Required	Pass
104	0.010	0.646	0.117	0.065	0.026	0.700	#13	0.080	Not Required	Pass
105	0.010	0.402	0.119	0.064	0.029	0.418	#13	0.074	Not Required	Pass
106	0.010	0.647	0.071	0.065	0.017	0.693	#13	0.045	Not Required	Pass
107	0.010	0.402	0.119	0.064	0.029	0.418	#13	0.074	Not Required	Pass
108	0.000	0.070	0.087	0.037	0.014	0.134	#21	0.095	Not Required	Pass
109	0.005	0.033	0.055	0.001	0.000	0.089	#13	0.204	Not Required	Pass
110	0.010	0.646	0.117	0.065	0.026	0.700	#13	0.080	Not Required	Pass
111	0.000	0.071	0.086	0.037	0.014	0.133	#21	0.095	Not Required	Pass
112	0.001	0.309	0.319	0.071	0.064	0.628	#13	0.035	Not Required	Pass
113	0.005	0.149	0.287	0.051	0.019	0.378	#21	0.286	Not Required	Pass
114	0.007	0.154	0.286	0.051	0.019	0.377	#21	0.286	Not Required	Pass
115	0.000	0.128	0.162	0.037	0.014	0.251	#21	0.346	Not Required	Pass
116	0.000	0.128	0.164	0.037	0.014	0.252	#21	0.346	Not Required	Pass
201	0.023	0.516	0.001	0.046	0.000	0.528	#13	0.350	Not Required	Pass
202	0.002	0.298	0.311	0.068	0.062	0.609	#13	0.035	Not Required	Pass
203	0.010	0.627	0.065	0.063	0.013	0.668	#13	0.045	Not Required	Pass
204	0.009	0.623	0.122	0.063	0.027	0.680	#13	0.080	Not Required	Pass
205	0.010	0.390	0.126	0.062	0.032	0.408	#13	0.074	Not Required	Pass
206	0.009	0.639	0.062	0.064	0.013	0.679	#13	0.045	Not Required	Pass
207	0.009	0.396	0.130	0.064	0.032	0.413	#13	0.074	Not Required	Pass
208	0.000	0.070	0.149	0.030	0.011	0.198	#21	Not Required	Not Required	Pass
209	0.007	0.038	0.059	0.001	0.000	0.099	#13	0.204	Not Required	Pass
210	0.009	0.636	0.128	0.064	0.027	0.688	#13	0.080	Not Required	Pass
211	0.000	0.070	0.149	0.030	0.011	0.198	#21	Not Required	Not Required	Pass
212	0.002	0.306	0.323	0.069	0.063	0.629	#13	0.035	Not Required	Pass
213	0.006	0.207	0.322	0.054	0.019	0.455	#21	0.190	Not Required	Pass
214	0.007	0.211	0.322	0.054	0.019	0.455	#21	0.286	Not Required	Pass
215	0.000	0.124	0.163	0.040	0.014	0.248	#21	0.346	Not Required	Pass
216	0.000	0.124	0.164	0.040	0.014	0.249	#21	0.346	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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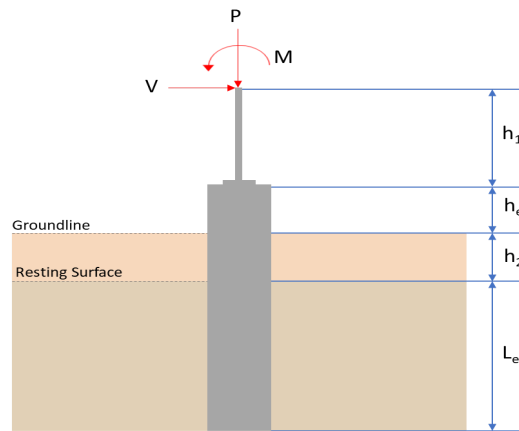
## SkyCiv Foundation Design

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

$L = 7.25$  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)	5.941	8.648
$V_x$ (kip)	-4.431	-7.387
$V_z$ (kip)	0.014	0.024
$M_x$ (kipft)	0.031	0.052
$M_z$ (kipft)	45.486	76.179

### 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,

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

$$H_o = \frac{(-4.431 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70557 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(45.486 \text{ kipft}) + ((-4.431 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.243 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.6719 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0022293 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.031 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0049363 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.7945 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(6.6719 \text{ ft}), (0.7945 \text{ ft})]$$

$$L_{e,req} = 6.672 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(6.672 \text{ ft})}{(7.25 \text{ ft})}$$

$$Ratio = 0.92028$$

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

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

$$q = \frac{(5.941 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37131 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37131 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18566$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.70557 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 7.243 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (7.243 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (7.243 \text{ kipft/ft})) + (4 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0267 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.243 \text{ kipft/ft})) + (3 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (7.243 \text{ kipft/ft})) + (2 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.2304 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (7.243 \text{ kipft/ft})) + ((-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0697 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0267 \text{ ft})}{2}$$

$$p_a = 0.37701 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2304 \text{ kip/ft}^2)}{(0.37701 \text{ kip/ft}^2)}$$

$$Ratio = 0.61113$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0697 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.98359$$

Status: **PASS**  
Ratio: **0.610**

Status: **PASS**  
Ratio: **0.980**

**Considering z-direction:**

$H_o = 0.0022293 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.0049363 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.0049363 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0049363 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2477 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0049363 \text{ kipft/ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0049363 \text{ kipft/ft})) + (2 \times (0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.0014094 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.0049363 \text{ kipft/ft})) + ((0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.0029719 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.2477 \text{ ft})}{2}$$

$$p_a = 0.39358 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0014094 \text{ kip/ft}^2)}{(0.39358 \text{ kip/ft}^2)}$$

$$Ratio = 0.003581$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

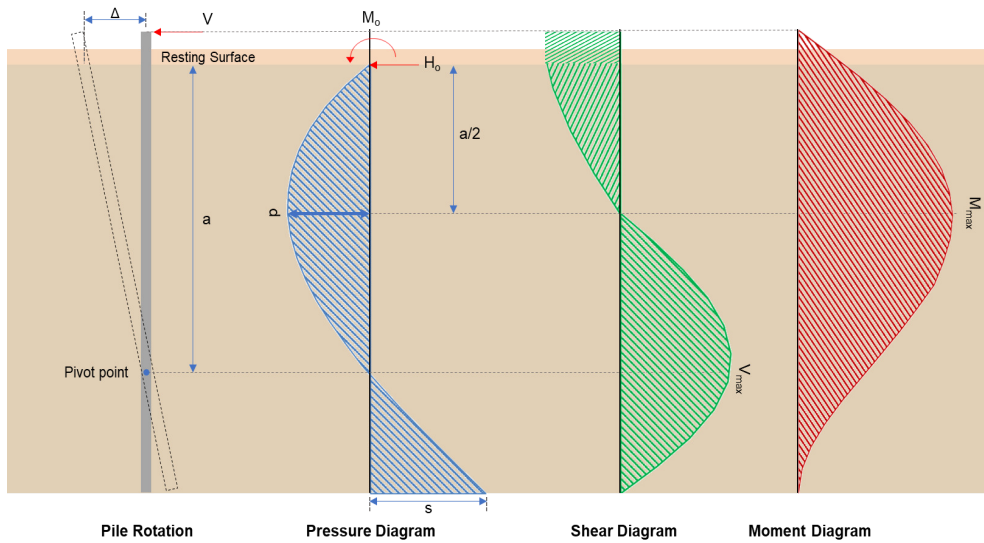
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0029719 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0027328$$

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,

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

$$H_o = \frac{(-7.387 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1763 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(76.179 \text{ kipft}) + ((-7.387 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.13 \text{ kipft/ft}$$

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

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

$$E = \frac{(12.13 \text{ kipft/ft})}{(-1.1763 \text{ kip/ft})}$$

$$E = 10.313 \text{ ft}$$

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

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

$$a = \frac{(4 \times (12.13 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times 12.13 \text{ kipft/ft}) + (4 \times (-1.1763 \text{ kip/ft}) \times 7.25 \text{ ft})}$$

$$a = \frac{(-1.1763 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (12.13 \text{ kipft/ft})) + (4 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0261 \text{ ft}$$

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

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

$$V_{max} = ((-1.1763 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0261 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0261 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.945 \text{ kip}$$

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

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

$$M_{max} = ((-1.1763 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(10.313 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.087 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.052 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0082803 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.0082803 \text{ kipft/ft})}{(0.0038217 \text{ kip/ft})}$$

$$E = 2.1667 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.0082803 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0082803 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2505 \text{ ft}$$

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

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

$$V_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.2505 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.2505 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.01835 \text{ kip}$$

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

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

$$M_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(2.1667 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.056695 \text{ kipft}$$

### 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,  $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.648 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.309 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.309 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(8.648 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0032327</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.648 \text{ kip} \rightarrow 8648 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8648 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.64 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.64 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.64 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.85 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 14.945 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.945 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.13483$$

Status: **PASS**  
Ratio: **0.130**

**Considering z-direction:**

$V_{max} = 0.01835 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.01835 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.00016554$$

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:**

$M_{max} = 51.087 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.087 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.20467$$

Status: **PASS**  
Ratio: **0.200**

**Considering z-direction:**

$M_{max} = 0.056695 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.056695 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00022714$$

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

REFERENCES	CALCULATIONS	RESULTS
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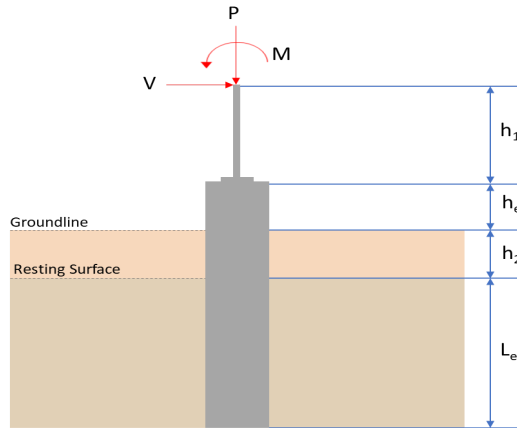
## SkyCiv Foundation Design

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

$L = 7.25$  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)	6.159	8.924
$V_x$ (kip)	-4.535	-7.559
$V_z$ (kip)	0.000	0.000
$M_x$ (kipft)	0.000	0.000
$M_z$ (kipft)	46.276	77.406

### 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,

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

$$H_o = \frac{(-4.535 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.72213 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(46.276 \text{ kipft}) + ((-4.535 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.3688 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.6912 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

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

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.6912 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.691 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

*Ratio* - Embedded depth

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

$$\text{Ratio} = \frac{(6.691 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.9229$$

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

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

$$q = \frac{(6.159 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38494 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(0.38494 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19247$$

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

Czerniak

### Lateral Soil Pressure (ASD):

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

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.72213 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 7.3688 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (7.3688 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.72213 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (7.3688 \text{ kipft/ft})) + (4 \times (-0.72213 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0275 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.3688 \text{ kipft/ft})) + (3 \times (-0.72213 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (7.3688 \text{ kipft/ft})) + (2 \times (-0.72213 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.23248 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (7.3688 \text{ kipft/ft})) + ((-0.72213 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0847 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0275 \text{ ft})}{2}$$

$$p_a = 0.37706 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23248 \text{ kip/ft}^2)}{(0.37706 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61656$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

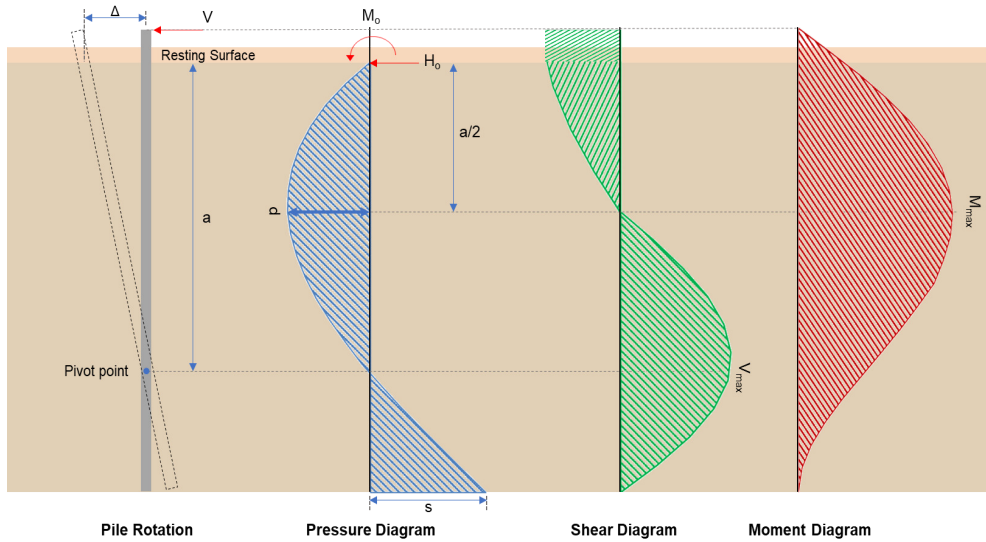
$$p_s = 1.0875 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0847 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-7.559 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2037 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(77.406 \text{ kipft}) + ((-7.559 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.326 \text{ kipft/ft}$$

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

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

$$E = \frac{(12.326 \text{ kipft/ft})}{(-1.2037 \text{ kip/ft})}$$

$$E = 10.24 \text{ ft}$$

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

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

$$a = \frac{(4 \times (12.326 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-1.2037 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (12.326 \text{ kipft/ft})) + (4 \times (-1.2037 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0271 \text{ ft}$$

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

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

$$V_{max} = ((-1.2037 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.24 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0271 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.24 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0271 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.208 \text{ kip}$$

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

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

$$M_{max} = ((-1.2037 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(10.24 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0271 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.24 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0271 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (10.24 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0271 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.969 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.924 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.3 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.3 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

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

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

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

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

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

**Ties:**

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

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

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

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

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

**Summary:**

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

Ties: **#3(0.375 in) - 10 in**

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

22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(8.924 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0033358$$

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

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

**Parameters:**

22.5.2.2  $b_w$  = 48 in - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,

22.5.5.1.1  $V_{c,max}$  - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.924 \text{ kip} \rightarrow 8924 \text{ lbf}$ ,

22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8924 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.68 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.68 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.68 \text{ kip}$$

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

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

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

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

$A_v$  - 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)

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

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

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

$V_s$  - 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

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

$$\phi V_n = (0.65) \times ((119.68 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.87 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 15.208 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.208 \text{ kip})}{(110.87 \text{ kip})}$$

$$\text{Ratio} = 0.13717$$

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

**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:** $M_{max} = 51.969 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(51.969 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.20821$$

Status: **PASS**  
Ratio: **0.210**

REFERENCES	CALCULATIONS	RESULTS
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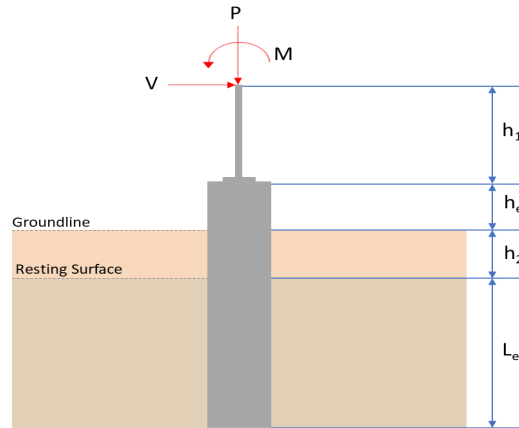
## SkyCiv Foundation Design

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

$L = 7.25$  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)	5.941	8.648
$V_x$ (kip)	-4.431	-7.387
$V_z$ (kip)	-0.014	-0.024
$M_x$ (kipft)	-0.031	-0.052
$M_z$ (kipft)	45.486	76.180

### 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,

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

$$H_o = \frac{(-4.431 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70557 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(45.486 \text{ kipft}) + ((-4.431 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.243 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.6719 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0022293 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.031 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0049363 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.67322 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.6719 \text{ ft}), (0.67322 \text{ ft})]$$

$$L_{e,req} = 6.672 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.672 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.92028$$

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

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

$$q = \frac{(5.941 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37131 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37131 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18566$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.70557 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 7.243 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (7.243 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (7.243 \text{ kipft/ft})) + (4 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0267 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.243 \text{ kipft/ft})) + (3 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (7.243 \text{ kipft/ft})) + (2 \times (-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.2304 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (7.243 \text{ kipft/ft})) + ((-0.70557 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0697 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0267 \text{ ft})}{2}$$

$$p_a = 0.37701 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.2304 \text{ kip/ft}^2)}{(0.37701 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61113$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0697 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98359$$

Status: **PASS**  
Ratio: **0.610**

Status: **PASS**  
Ratio: **0.980**

#### Considering z-direction:

$H_o = -0.0022293 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.0049363 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.0049363 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0049363 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2477 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0049363 \text{ kipft/ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.0049363 \text{ kipft/ft})) + (2 \times (-0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.00067296 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.0049363 \text{ kipft/ft})) + ((-0.0022293 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.00071798 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.2477 \text{ ft})}{2}$$

$$p_a = 0.39358 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00067296 \text{ kip/ft}^2)}{(0.39358 \text{ kip/ft}^2)}$$

$$Ratio = -0.0017099$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

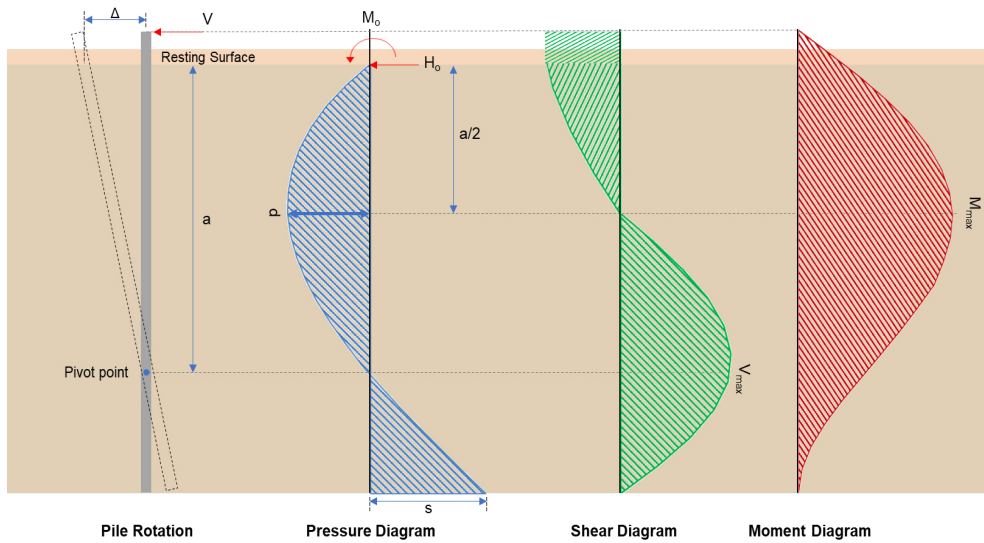
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00071798 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00066021$$

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,

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

$$H_o = \frac{(-7.387 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1763 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(76.18 \text{ kipft}) + ((-7.387 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 12.131 \text{ kipft/ft}$$

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

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

$$E = \frac{(12.131 \text{ kipft/ft})}{(-1.1763 \text{ kip/ft})}$$

$$E = 10.313 \text{ ft}$$

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

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

$$a = \frac{(4 \times (12.131 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{6 \times (12.131 \text{ kipft/ft}) + 4 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft})}$$

$$a = \frac{(6 \times (12.131 \text{ kipft/ft})) + (4 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft}))}{(6 \times (12.131 \text{ kipft/ft})) + (4 \times (-1.1763 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0261 \text{ ft}$$

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

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

$$V_{max} = ((-1.1763 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0261 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0261 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 14.945 \text{ kip}$$

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

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

$$M_{max} = ((-1.1763 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(10.313 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.313 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.0261 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.087 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.052 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0082803 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.0082803 \text{ kipft/ft})}{(-0.0038217 \text{ kip/ft})}$$

$$E = 2.1667 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.0082803 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0082803 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2505 \text{ ft}$$

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

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

$$V_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.2505 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.2505 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.01835 \text{ kip}$$

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

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

$$M_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (7.25 \text{ ft})) \times \left[ \left( \frac{(2.1667 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left( \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.1667 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left( \frac{(5.2505 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.056695 \text{ kipft}$$

### 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,  $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.648 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.309 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.309 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <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}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.648 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0032327$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.648 \text{ kip} \rightarrow 8648 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8648 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.64 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.64 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.64 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.64 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.85 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 14.945 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(14.945 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.13483$$

Status: **PASS**  
Ratio: **0.130**

**Considering z-direction:**

$V_{max} = 0.01835 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.01835 \text{ kip})}{(110.85 \text{ kip})}$$

$$Ratio = 0.00016554$$

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:**

$M_{max} = 51.087 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.087 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.20468$$

Status: **PASS**  
Ratio: **0.200**

**Considering z-direction:**

$M_{max} = 0.056695 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.056695 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00022714$$

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