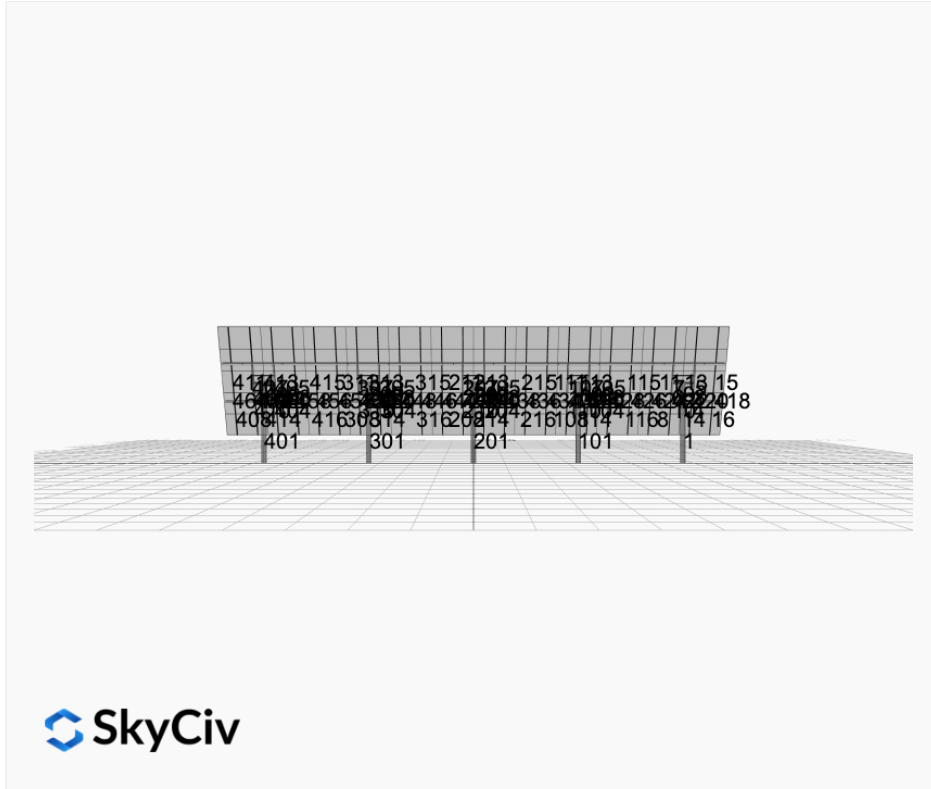


**Project Name:** Giesey Opt1&2 - V1Jb **Date:** Fri Aug 01 2025  
**Location:** 8385 Jordan Rd, Melvin, MI 48454, USA **Number of Modules:** 60  
**Unique ID:** 5P-19.75-10TOP-SD-45-L-5Hx12W- **Number of Poles:** 5  
 HH5C **Date Sold:**  
**Dealer:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	21.58 ft
<b>Array Dimensions E/W</b>	94.90 ft
<b>Winter Tilt Angle</b>	70
<b>Front Edge Clearance</b>	5 ft

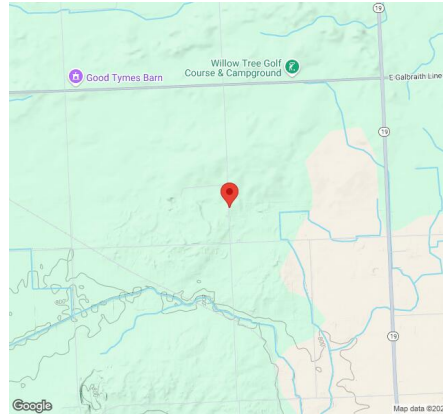
### MT Solar Bill of Materials (5P-19.75-10TOP-SD-45-L-5Hx12W-HH5C)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	5
MTS-HF-SD	H-Frame Assembly-SD	5
MTS-SD-Wing-45	45IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	8
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

### Rail Bill of Materials

Part	Qty
Rails (259in)	24
Rail Attachment	96
Module Mid Clamp	96
Module End Clamp	48
Ground Lug	12

## Site Details:



**Site Address:** 8385 Jordan Rd, Melvin, MI 48454, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	51.30 in
<b>Module Length:</b>	93.90in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	12
<b>Total Number of Modules:</b>	60
<b>Winter Tilt Angle:</b>	70
<b>Front Edge Clearance:</b>	5
<b>Total Array Height at Tilt:</b>	25.28 ft
<b>Total Frame Length:</b>	94.00 ft
<b>Module Info/Notes:</b>	Risen RSM132-8-720-740BHDG Thornova Solar TS-BGT66(700-720)-G12
<b>Array Dimensions N/S:</b>	21.58 ft
<b>Array Dimensions E/W:</b>	94.90 ft
<b>Rail Length:</b>	259.00 in
<b>Rail Spacing:</b>	3.95 ft

### Support Specifications

<b>Pole Size:</b>	10in Pipe Sch 40
<b>Pole Length above Grade:</b>	15.14 ft
<b>Number of Poles:</b>	5
<b>Pole Spacing:</b>	19.75 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 8.50 ft Pile 2: 9.00 ft Pile 3: 9.00 ft Pile 4: 9.00 ft Pile 5: 8.50 ft
<b>Foundation Volume:</b>	26.074 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	8385 Jordan Rd, Melvin, MI 48454, USA

<b>Wind Speed:</b>	101 mph
<b>Snow Load:</b>	25 psf

### **Design Disclaimer**

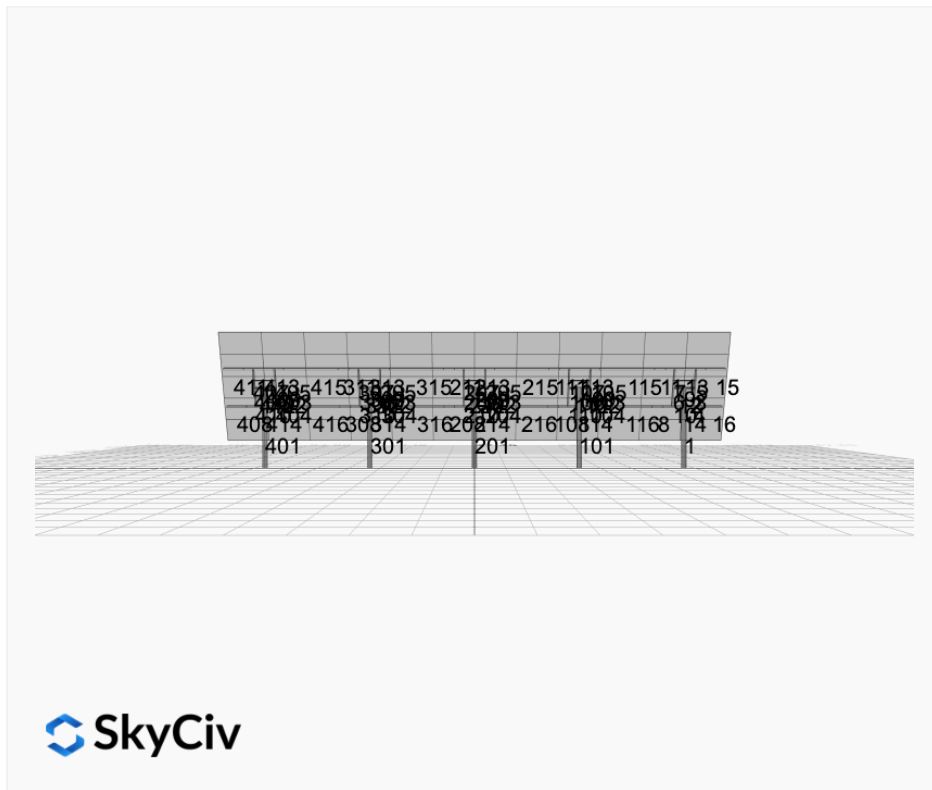
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

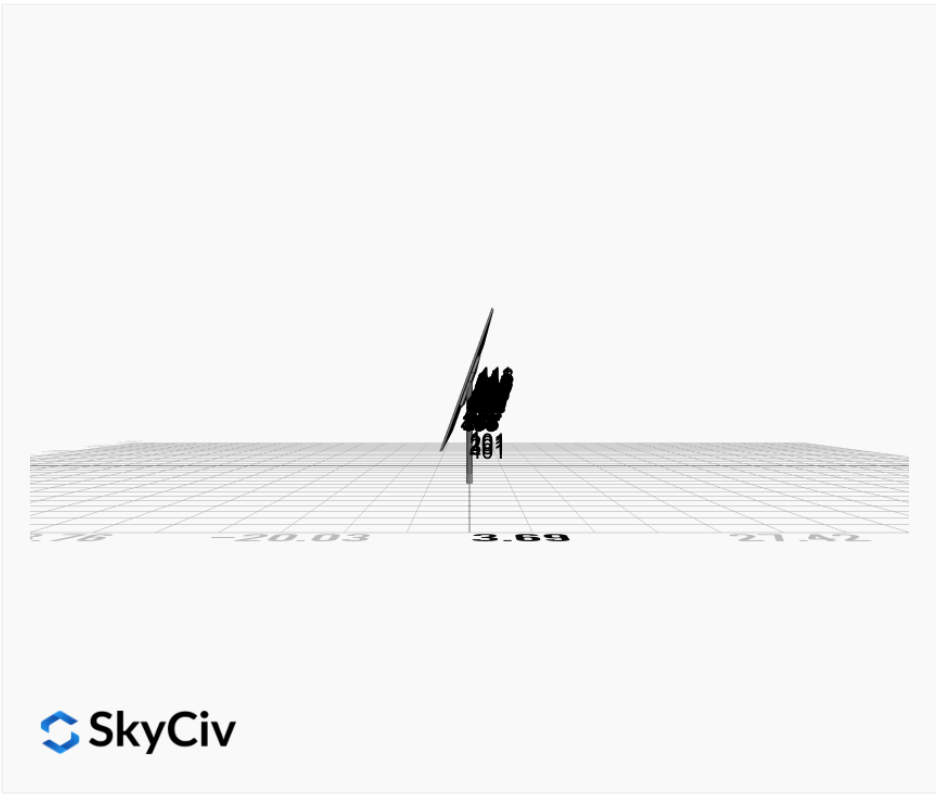
## AutoDesigner Input

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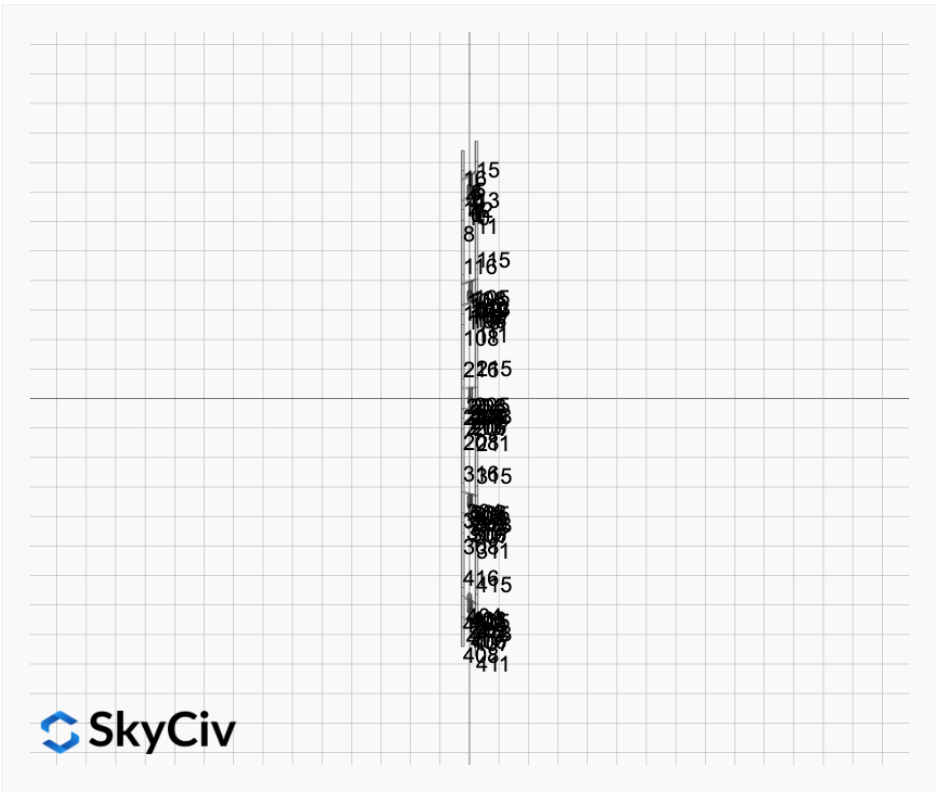
## Design Notes:

- 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

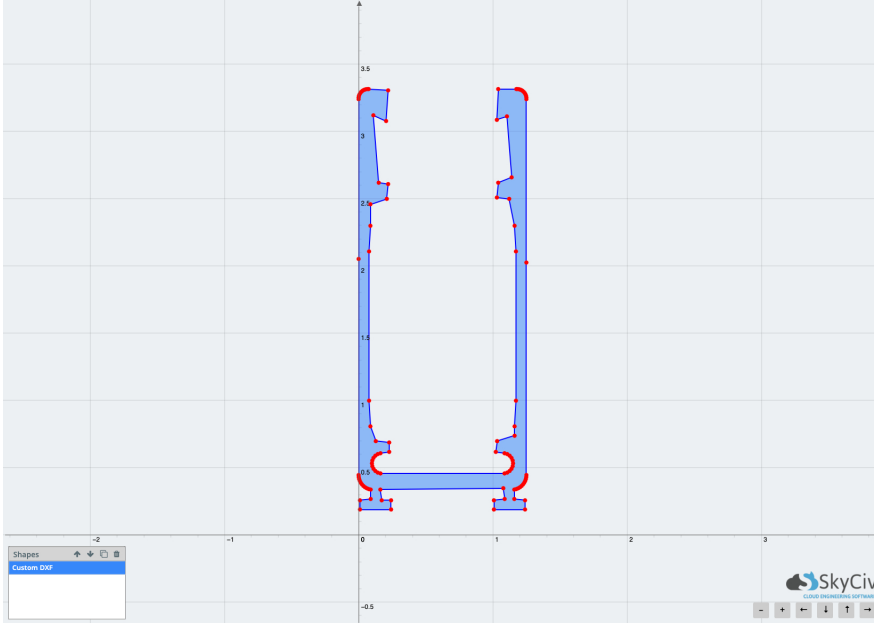


 SkyCiv



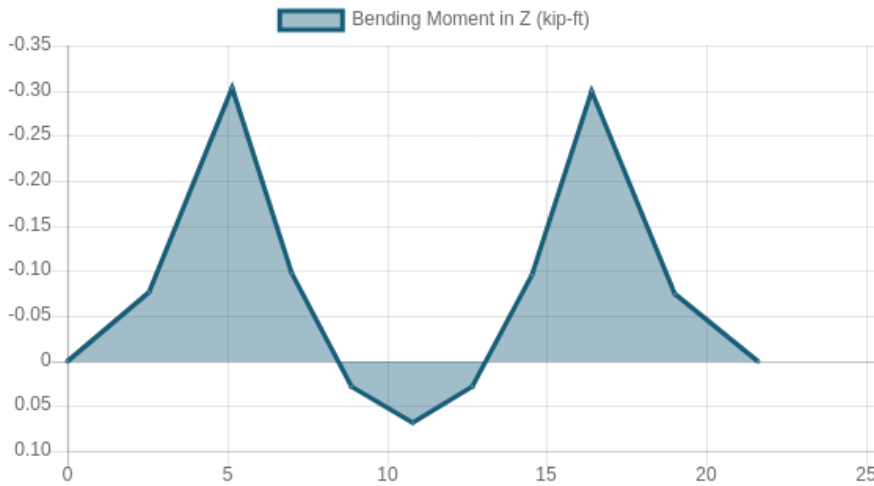
### Rail Design Check

**Rail Length:** 21.583333333333332 ft  
**Additional Restraints Required:** 4ft Spread Clamps  
**Tributary Width:** 3.954166666666667 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Wind uplift Case A (X):** 0.0000 kip/ft  
**Wind uplift Case A (Y):** 0.0796 kip/ft  
**Wind downforce Case A:** -0.0796 kip/ft  
**Dead (Panel load):** -0.0796 kip/ft

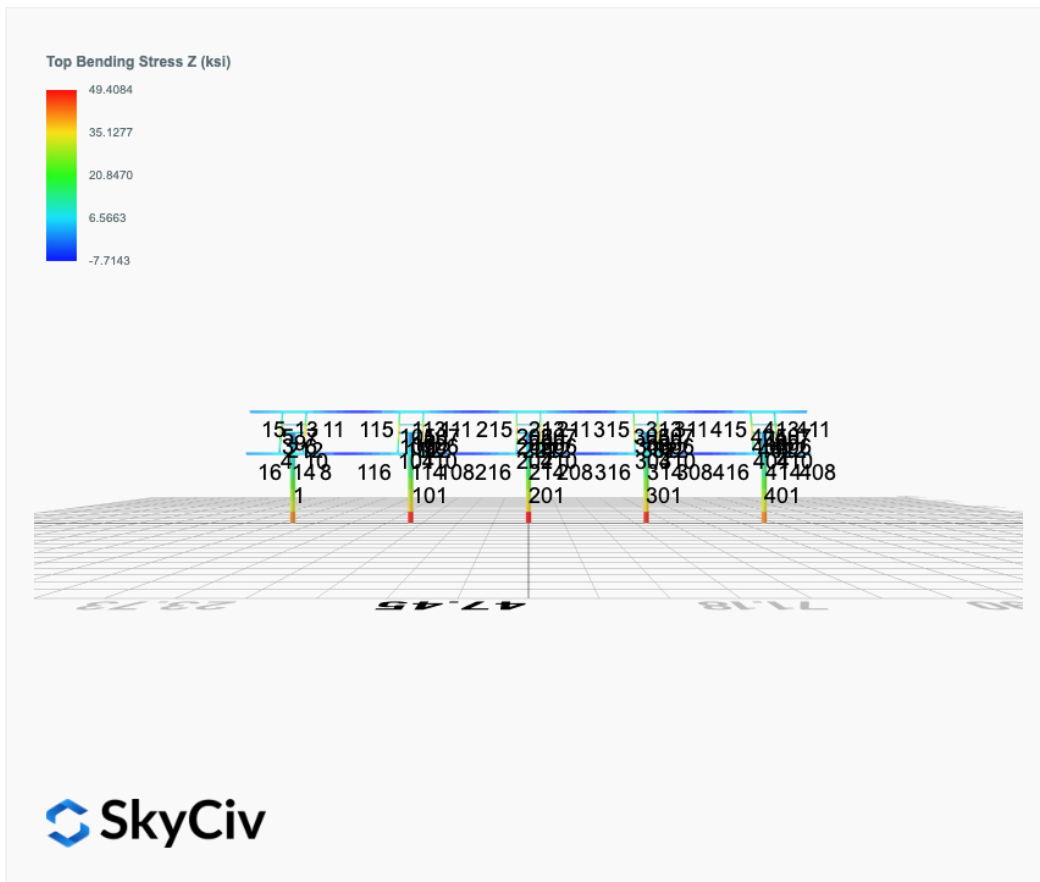
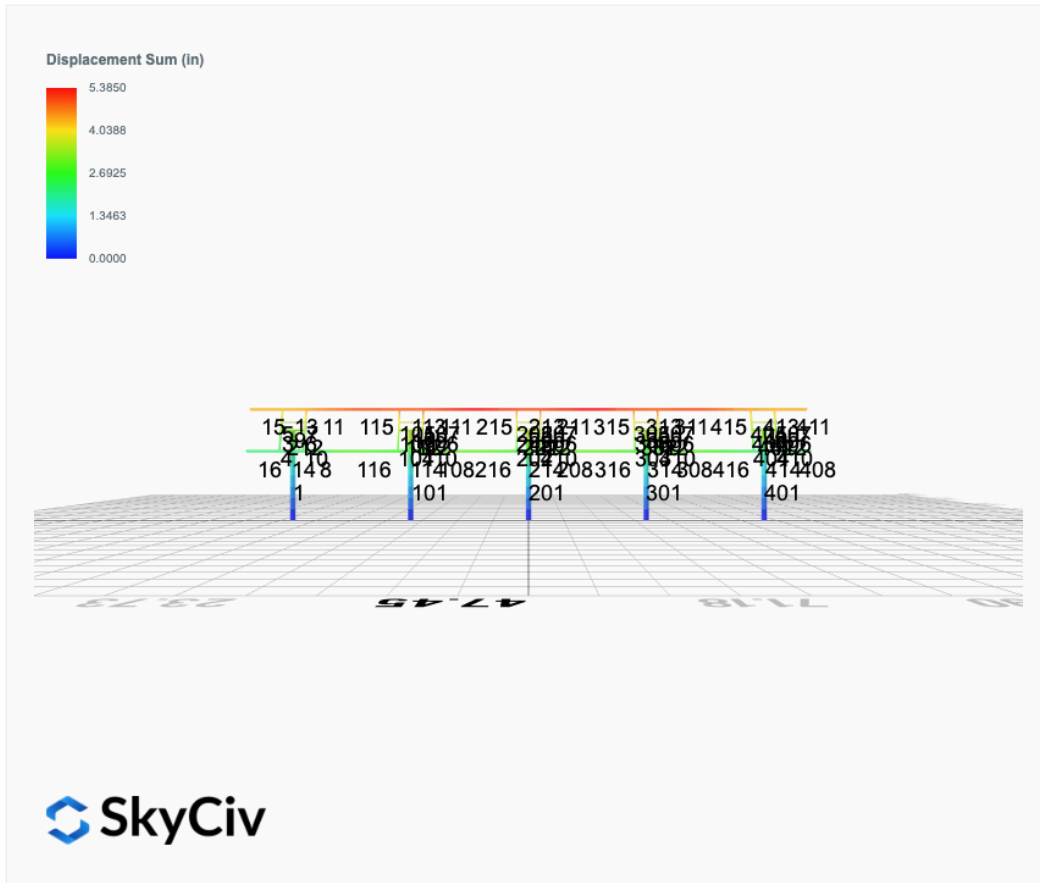


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	28.51910814	0.827	PASS
Material Yield	34.5	28.51910814	0.827	PASS
Material Strength	37	28.51910814	0.771	PASS

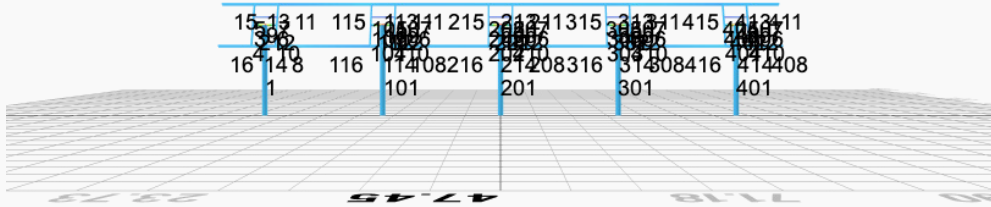
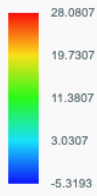
Member 1, ULS: 1.14D



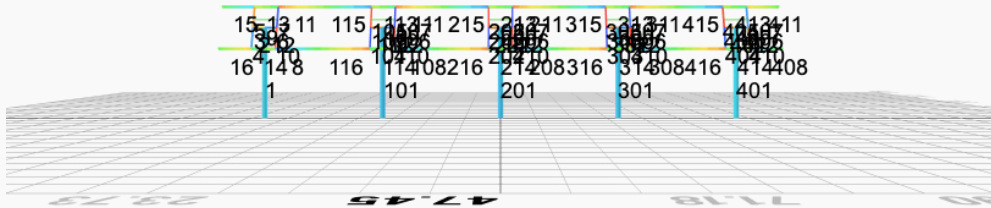
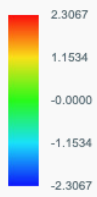
# FEM Results (Envelope Worst Case for each member)



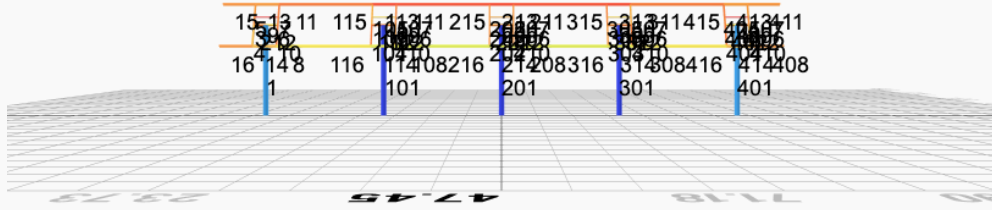
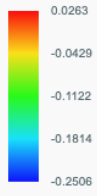
Top Bending Stress Y (ksi)



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	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 2. D + L	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 3. D + (S or Lr or R)	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 3. D + (S or Lr or R)	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 5b. D + 0.7E	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 8. 0.6D + 0.7E	0.0032	1.6342	0.0188	0.0865	-0.0095	-0.0408
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.3048	4.2776	0.0836	0.3553	-1.0979	65.6861
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.3138	1.1701	-0.0203	-0.0640	1.0521	-64.9958
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2273	3.8891	0.0705	0.3025	-0.8274	49.2476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2367	1.5585	-0.0074	-0.0119	0.7851	-48.7639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2273	3.8891	0.0705	0.3025	-0.8274	49.2476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2367	1.5585	-0.0074	-0.0119	0.7851	-48.7639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0053	2.7237	0.0313	0.1442	-0.0159	-0.0680
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.3069	3.1881	0.0711	0.2977	-1.0916	65.7132
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0032	1.6342	0.0188	0.0865	-0.0095	-0.0408
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.3117	0.0806	-0.0328	-0.1216	1.0585	-64.9687
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0032	1.6342	0.0188	0.0865	-0.0095	-0.0408

### 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	5.8584
Shear X	-7.1885
Shear Z	0.1252
Moment X	0.5271
Moment Y (Twist)	1.8322
Moment Z	110.4610

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.2776
Shear X	-4.3138
Shear Z	0.0836
Moment X	0.3553
Moment Y (Twist)	1.0979
Moment Z	65.7132

## 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	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 2. D + L	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 3. D + (S or Lr or R)	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 3. D + (S or Lr or R)	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 5b. D + 0.7E	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 8. 0.6D + 0.7E	-0.0029	1.7981	-0.0002	-0.0013	0.0037	0.0493
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.8689	4.7805	0.0083	0.0303	-0.2418	74.1395
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.8608	1.2129	-0.0085	-0.0326	0.2433	-72.9714
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6529	4.3346	0.0062	0.0222	-0.1798	55.6251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6444	1.6589	-0.0065	-0.0250	0.1840	-54.7080
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6529	4.3346	0.0062	0.0222	-0.1798	55.6251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6444	1.6589	-0.0065	-0.0250	0.1840	-54.7080
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0049	2.9969	-0.0003	-0.0021	0.0062	0.0822
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.8670	3.5817	0.0085	0.0312	-0.2443	74.1066
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0029	1.7981	-0.0002	-0.0013	0.0037	0.0493
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.8627	0.0142	-0.0084	-0.0318	0.2408	-73.0043
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0029	1.7981	-0.0002	-0.0013	0.0037	0.0493

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.5687
Shear X	-8.1110
Shear Z	-0.0145
Moment X	-0.0548
Moment Y (Twist)	0.4157
Moment Z	124.6574

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.7805
Shear X	-4.8689
Shear Z	-0.0085
Moment X	-0.0326
Moment Y (Twist)	0.2443
Moment Z	74.1395

### 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	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 2. D + L	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 3. D + (S or Lr or R)	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 3. D + (S or Lr or R)	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 5b. D + 0.7E	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 8. 0.6D + 0.7E	-0.0005	1.7968	0.0000	-0.0000	0.0000	0.0161
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.9125	4.7857	0.0000	-0.0000	0.0000	74.9538
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.9107	1.2040	0.0000	-0.0000	0.0000	-73.8487
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269

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	-3.6846	4.3380	0.0000	-0.0000	0.0000	56.2220
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6828	1.6517	0.0000	-0.0000	0.0000	-55.3798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6846	4.3380	0.0000	-0.0000	0.0000	56.2220
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6828	1.6517	0.0000	-0.0000	0.0000	-55.3798
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0008	2.9947	0.0000	-0.0000	0.0000	0.0269
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.9122	3.5878	0.0000	-0.0000	0.0000	74.9430
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0005	1.7968	0.0000	-0.0000	0.0000	0.0161
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.9110	0.0061	0.0000	-0.0000	0.0000	-73.8594
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0005	1.7968	0.0000	-0.0000	0.0000	0.0161

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	6.5790
Shear X	-8.1876
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	126.0869

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.7857
Shear X	-4.9125
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	74.9538

### Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

#### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 2. D + L	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 3. D + (S or Lr or R)	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 3. D + (S or Lr or R)	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 5b. D + 0.7E	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 8. 0.6D + 0.7E	-0.0029	1.7981	0.0002	0.0013	-0.0037	0.0493
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.8689	4.7805	-0.0083	-0.0303	0.2418	74.1395
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.8608	1.2129	0.0085	0.0326	-0.2433	-72.9714
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6529	4.3346	-0.0062	-0.0222	0.1798	55.6251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6444	1.6589	0.0065	0.0250	-0.1840	-54.7080
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.6529	4.3346	-0.0062	-0.0222	0.1798	55.6251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.6444	1.6589	0.0065	0.0250	-0.1840	-54.7080
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0049	2.9969	0.0003	0.0021	-0.0062	0.0822

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.8670	3.5817	-0.0085	-0.0312	0.2443	74.1066
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0029	1.7981	0.0002	0.0013	-0.0037	0.0493
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.8627	0.0142	0.0084	0.0318	-0.2408	-73.0043
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0029	1.7981	0.0002	0.0013	-0.0037	0.0493

### Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

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

Result	Value (kip, kip-ft)
Axial	6.5687
Shear X	-8.1110
Shear Z	0.0145
Moment X	0.0548
Moment Y (Twist)	0.4159
Moment Z	124.6574

### Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	4.7805
Shear X	-4.8689
Shear Z	0.0085
Moment X	0.0326
Moment Y (Twist)	0.2443
Moment Z	74.1395

## Reaction Forces for Foundation 5 (Node ID#401), (kip, kip-ft)

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 2. D + L	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 3. D + (S or Lr or R)	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 3. D + (S or Lr or R)	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 5b. D + 0.7E	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 8. 0.6D + 0.7E	0.0032	1.6342	-0.0188	-0.0865	0.0095	-0.0408
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.3048	4.2776	-0.0836	-0.3554	1.0979	65.6861
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.3138	1.1701	0.0203	0.0639	-1.0521	-64.9958
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2273	3.8891	-0.0705	-0.3026	0.8274	49.2476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2367	1.5585	0.0074	0.0119	-0.7851	-48.7639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2273	3.8891	-0.0705	-0.3026	0.8274	49.2476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2367	1.5585	0.0074	0.0119	-0.7851	-48.7639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0053	2.7237	-0.0313	-0.1442	0.0159	-0.0679
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.3069	3.1881	-0.0711	-0.2977	1.0916	65.7133
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0032	1.6342	-0.0188	-0.0865	0.0095	-0.0408
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.3117	0.0806	0.0328	0.1216	-1.0585	-64.9686
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0032	1.6342	-0.0188	-0.0865	0.0095	-0.0408

### 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.

### 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	5.8583
Shear X	-7.1885
Shear Z	-0.1252
Moment X	-0.5272
Moment Y (Twist)	1.8323
Moment Z	110.4619

Result	Value (kip, kip-ft)
Axial	4.2776
Shear X	-4.3138
Shear Z	-0.0836
Moment X	-0.3554
Moment Y (Twist)	1.0979
Moment Z	65.7133

## 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)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
11	10in Pipe Sch 40	10.75	0.36				

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

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

Section Properties								
ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )







104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.36	6.45	30.09	45.74
114	120.60	84.03	18.33	6.45	30.09	45.74
115	120.60	68.63	15.32	6.45	30.09	45.74
116	120.60	68.63	15.57	6.45	30.09	45.74
201	535.87	243.54	147.68	147.68	160.76	160.76
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	10.99	6.26	29.14	16.61
208	120.60	115.40	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.84	10.99	6.26	29.14	16.61
211	120.60	115.40	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.03	18.27	6.45	30.09	45.74
214	120.60	84.03	18.26	6.45	30.09	45.74
215	120.60	68.63	15.75	6.45	30.09	45.74
216	120.60	68.63	15.79	6.45	30.09	45.74
301	535.87	243.54	147.68	147.68	160.76	160.76
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.89	10.99	6.26	29.14	16.61
304	79.65	72.84	10.99	6.26	29.14	16.61
305	79.65	74.30	10.99	6.26	29.14	16.61
306	79.65	74.89	10.99	6.26	29.14	16.61
307	79.65	74.30	10.99	6.26	29.14	16.61
308	120.60	115.40	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.84	10.99	6.26	29.14	16.61
311	120.60	115.40	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	84.03	18.36	6.45	30.09	45.74
314	120.60	84.03	18.33	6.45	30.09	45.74
315	120.60	68.63	15.57	6.45	30.09	45.74
316	120.60	68.63	15.68	6.45	30.09	45.74
401	535.87	243.54	147.68	147.68	160.76	160.76
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.89	10.99	6.26	29.14	16.61
404	79.65	72.84	10.99	6.26	29.14	16.61
405	79.65	74.30	10.99	6.26	29.14	16.61
406	79.65	74.89	10.99	6.26	29.14	16.61
407	79.65	74.30	10.99	6.26	29.14	16.61

407	79.03	74.30	10.99	0.20	29.14	10.01
408	120.60	54.44	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.84	10.99	6.26	29.14	16.61
411	120.60	54.44	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	84.03	18.84	6.45	30.09	45.74
414	120.60	84.03	18.97	6.45	30.09	45.74
415	120.60	68.63	15.44	6.45	30.09	45.74
416	120.60	68.63	15.40	6.45	30.09	45.74

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.024	0.748	0.009	0.045	0.001	0.764	#13	0.519	Not Required	Pass
2	0.002	0.204	0.360	0.052	0.076	0.565	#13	0.034	Not Required	Pass
3	0.008	0.658	0.051	0.065	0.006	0.707	#13	0.044	Not Required	Pass
4	0.007	0.657	0.125	0.066	0.021	0.751	#13	0.078	Not Required	Pass
5	0.008	0.409	0.127	0.066	0.025	0.433	#13	0.073	Not Required	Pass
6	0.010	0.806	0.085	0.082	0.015	0.884	#13	0.044	Not Required	Pass
7	0.010	0.501	0.171	0.080	0.034	0.537	#13	0.073	Not Required	Pass
8	0.002	0.085	0.084	0.049	0.010	0.125	#13	0.088	Not Required	Pass
9	0.007	0.048	0.102	0.003	0.003	0.131	#13	0.198	Not Required	Pass
10	0.010	0.785	0.162	0.079	0.028	0.868	#13	0.078	Not Required	Pass
11	0.001	0.079	0.085	0.050	0.010	0.116	#13	0.088	Not Required	Pass
12	0.002	0.289	0.474	0.067	0.093	0.762	#13	0.034	Not Required	Pass
13	0.003	0.218	0.227	0.064	0.013	0.380	#13	0.265	Not Required	Pass
14	0.004	0.214	0.225	0.063	0.013	0.359	#13	0.177	Not Required	Pass
15	0.000	0.073	0.081	0.030	0.006	0.143	#13	Not Required	Not Required	Pass
16	0.000	0.073	0.081	0.030	0.006	0.143	#13	Not Required	Not Required	Pass
101	0.027	0.844	0.001	0.050	0.000	0.858	#13	0.519	Not Required	Pass
102	0.002	0.274	0.456	0.068	0.093	0.731	#13	0.034	Not Required	Pass
103	0.010	0.805	0.077	0.080	0.011	0.876	#13	0.044	Not Required	Pass
104	0.010	0.816	0.163	0.082	0.028	0.927	#13	0.078	Not Required	Pass
105	0.010	0.500	0.171	0.080	0.034	0.538	#13	0.073	Not Required	Pass
106	0.010	0.839	0.076	0.084	0.011	0.905	#13	0.044	Not Required	Pass
107	0.010	0.522	0.170	0.084	0.034	0.559	#13	0.073	Not Required	Pass
108	0.002	0.068	0.084	0.049	0.010	0.111	#13	0.088	Not Required	Pass
109	0.009	0.034	0.088	0.001	0.000	0.120	#13	0.198	Not Required	Pass
110	0.010	0.835	0.163	0.084	0.028	0.935	#13	0.078	Not Required	Pass
111	0.002	0.073	0.086	0.049	0.010	0.106	#13	0.088	Not Required	Pass
112	0.002	0.284	0.482	0.068	0.097	0.767	#13	0.034	Not Required	Pass
113	0.003	0.212	0.227	0.063	0.013	0.393	#13	0.265	Not Required	Pass
114	0.005	0.230	0.226	0.064	0.013	0.410	#13	0.265	Not Required	Pass
115	0.002	0.255	0.131	0.049	0.010	0.368	#13	0.439	Not Required	Pass
116	0.002	0.251	0.132	0.050	0.010	0.365	#13	0.439	Not Required	Pass
201	0.027	0.854	0.000	0.051	0.000	0.867	#13	0.519	Not Required	Pass
202	0.002	0.280	0.475	0.068	0.096	0.756	#13	0.034	Not Required	Pass
203	0.010	0.834	0.077	0.083	0.011	0.903	#13	0.044	Not Required	Pass
204	0.010	0.827	0.163	0.083	0.028	0.930	#13	0.078	Not Required	Pass
205	0.010	0.518	0.170	0.083	0.034	0.555	#13	0.073	Not Required	Pass

206	0.010	0.834	0.077	0.083	0.011	0.903	#13	0.044	Not Required	Pass
207	0.010	0.518	0.170	0.083	0.034	0.555	#13	0.073	Not Required	Pass
208	0.002	0.066	0.084	0.050	0.010	0.118	#13	0.088	Not Required	Pass
209	0.009	0.033	0.087	0.001	0.000	0.114	#13	0.198	Not Required	Pass
210	0.010	0.827	0.163	0.083	0.028	0.930	#13	0.078	Not Required	Pass
211	0.002	0.068	0.086	0.050	0.010	0.117	#13	0.088	Not Required	Pass
212	0.002	0.280	0.475	0.068	0.096	0.756	#13	0.034	Not Required	Pass
213	0.003	0.233	0.227	0.064	0.013	0.414	#13	0.265	Not Required	Pass
214	0.006	0.240	0.225	0.064	0.013	0.415	#13	0.265	Not Required	Pass
215	0.003	0.232	0.131	0.050	0.010	0.345	#13	0.439	Not Required	Pass
216	0.002	0.226	0.132	0.050	0.010	0.340	#13	0.439	Not Required	Pass
301	0.027	0.844	0.001	0.050	0.000	0.858	#13	0.519	Not Required	Pass
302	0.002	0.284	0.482	0.068	0.097	0.767	#13	0.034	Not Required	Pass
303	0.010	0.839	0.076	0.084	0.011	0.905	#13	0.044	Not Required	Pass
304	0.010	0.835	0.163	0.084	0.028	0.935	#13	0.078	Not Required	Pass
305	0.010	0.522	0.170	0.084	0.034	0.559	#13	0.073	Not Required	Pass
306	0.010	0.805	0.077	0.080	0.011	0.876	#13	0.044	Not Required	Pass
307	0.010	0.500	0.171	0.080	0.034	0.538	#13	0.073	Not Required	Pass
308	0.002	0.079	0.084	0.050	0.010	0.116	#13	0.088	Not Required	Pass
309	0.009	0.034	0.088	0.001	0.000	0.120	#13	0.198	Not Required	Pass
310	0.010	0.816	0.163	0.082	0.028	0.927	#13	0.078	Not Required	Pass
311	0.001	0.087	0.086	0.049	0.010	0.111	#15	0.088	Not Required	Pass
312	0.002	0.274	0.456	0.068	0.093	0.731	#13	0.034	Not Required	Pass
313	0.003	0.212	0.227	0.063	0.013	0.393	#13	0.265	Not Required	Pass
314	0.005	0.230	0.226	0.064	0.013	0.410	#13	0.265	Not Required	Pass
315	0.003	0.234	0.131	0.049	0.010	0.346	#13	0.439	Not Required	Pass
316	0.002	0.227	0.132	0.049	0.010	0.341	#13	0.439	Not Required	Pass
401	0.024	0.748	0.009	0.045	0.001	0.764	#13	0.519	Not Required	Pass
402	0.002	0.289	0.474	0.067	0.093	0.762	#13	0.034	Not Required	Pass
403	0.010	0.806	0.085	0.082	0.015	0.884	#13	0.044	Not Required	Pass
404	0.010	0.785	0.162	0.079	0.028	0.868	#13	0.078	Not Required	Pass
405	0.010	0.501	0.171	0.080	0.034	0.537	#13	0.073	Not Required	Pass
406	0.008	0.658	0.051	0.065	0.006	0.707	#13	0.044	Not Required	Pass
407	0.008	0.409	0.127	0.066	0.025	0.433	#13	0.073	Not Required	Pass
408	0.000	0.073	0.081	0.030	0.006	0.143	#13	Not Required	Not Required	Pass
409	0.007	0.048	0.102	0.003	0.003	0.131	#13	0.198	Not Required	Pass
410	0.007	0.657	0.125	0.066	0.021	0.751	#13	0.078	Not Required	Pass
411	0.000	0.073	0.081	0.030	0.006	0.143	#13	Not Required	Not Required	Pass
412	0.002	0.204	0.360	0.052	0.076	0.565	#13	0.034	Not Required	Pass
413	0.003	0.218	0.227	0.064	0.013	0.380	#13	0.177	Not Required	Pass
414	0.004	0.214	0.225	0.063	0.013	0.359	#13	0.265	Not Required	Pass
415	0.002	0.252	0.131	0.050	0.010	0.364	#13	0.439	Not Required	Pass
416	0.002	0.253	0.132	0.049	0.010	0.366	#13	0.439	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
$F_y$	Specified minimum yield stress
$F_u$	Specified minimum tensile strength

A	Cross-sectional area
J	Torsional constant
$I_{yp}$	Moment of inertia about the Y axes
$I_{zp}$	Moment of inertia about the Z axes
$I_w$	Warping constant
$S_{yp}$	Plastic section modulus about the Y axis
$S_{zp}$	Plastic section modulus about the Z axis
KL	Effective length
$C_b$	Buckling modification factor (from all load combinations)
$L_b$	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
$P_n$	Nominal axial strength (tension/compression)
$M_n$	Nominal flexural strength (about Z/Y axis)
$V_n$	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
$M_z$	Design ratio in case of bending about Z axis
$M_y$	Design ratio in case of bending about Y axis
$V_y$	Design ratio in case of shear along Y axis
$V_z$	Design ratio in case of shear along Z axis
$(P, M_z, M_y)$	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided





REFERENCES	CALCULATIONS	RESULTS
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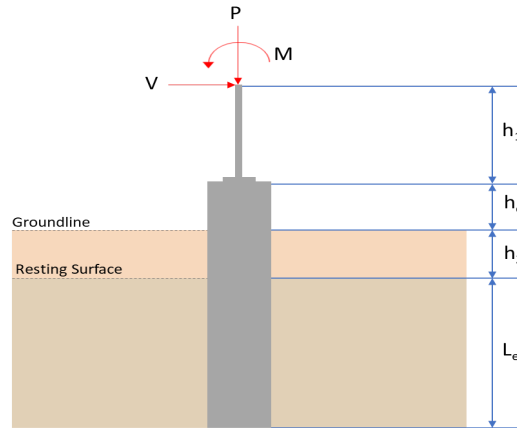
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 8.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.278	5.858
$V_x$ (kip)	-4.314	-7.189
$V_z$ (kip)	0.084	0.125
$M_x$ (kipft)	0.355	0.527
$M_z$ (kipft)	65.713	110.461

### 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,

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

$$H_o = \frac{(-4.314 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68694 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(65.713 \text{ kipft}) + ((-4.314 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.464 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.9801 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.084 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.013376 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.355 \text{ kipft}) + ((0.084 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.056529 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.8151 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.9801 \text{ ft}), (1.8151 \text{ ft})]$$

$$L_{e,req} = 7.98 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.98 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.93882$$

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

### 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

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

$$q = \frac{(4.278 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.26737 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.26737 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.13369$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.68694 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 10.464 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (10.464 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (10.464 \text{ kipft/ft})) + (4 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8587 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (10.464 \text{ kipft/ft})) + (3 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (10.464 \text{ kipft/ft})) + (2 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.31192 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (10.464 \text{ kipft/ft})) + ((-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.253 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8587 \text{ ft})}{2}$$

$$p_a = 0.4394 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.31192 \text{ kip/ft}^2)}{(0.4394 \text{ kip/ft}^2)}$$

$$Ratio = 0.70987$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.253 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.98278$$

Status: **PASS**  
Ratio: **0.710**

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

#### Considering z-direction:

$H_o = 0.013376 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.056529 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.056529 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.013376 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.056529 \text{ kipft/ft})) + (4 \times (0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0724 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.056529 \text{ kipft/ft})) + (3 \times (0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.056529 \text{ kipft/ft})) + (2 \times (0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.0084126 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.056529 \text{ kipft/ft})) + ((0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.018831 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0724 \text{ ft})}{2}$$

$$p_a = 0.45543 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0084126 \text{ kip/ft}^2)}{(0.45543 \text{ kip/ft}^2)}$$

$$Ratio = 0.018472$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

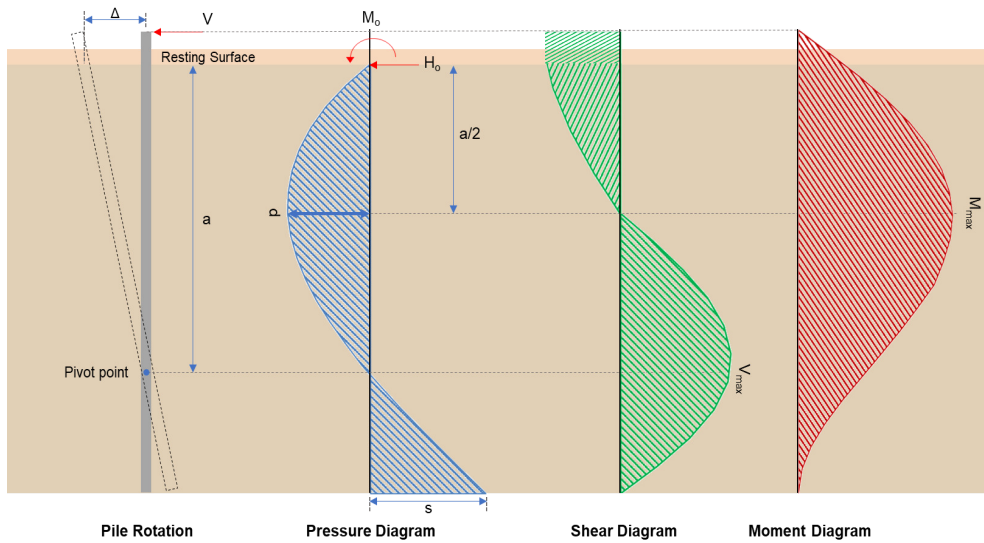
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.018831 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.014769$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-7.189 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1447 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(110.46 \text{ kipft}) + ((-7.189 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 17.589 \text{ kipft/ft}$$

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

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

$$E = \frac{(17.589 \text{ kipft/ft})}{(-1.1447 \text{ kip/ft})}$$

$$E = 15.365 \text{ ft}$$

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

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

$$a = \frac{(4 \times (17.589 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (17.589 \text{ kipft/ft})) + (4 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = \frac{(6 \times (17.589 \text{ kipft/ft})) + (4 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft}))}{}$$

$$a = 5.8575 \text{ ft}$$

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

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

$$V_{max} = ((-1.1447 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.8575 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.8575 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.668 \text{ kip}$$

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

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

$$M_{max} = ((-1.1447 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(15.365 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8575 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.8575 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.8575 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 71.552 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.125 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.019904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.527 \text{ kipft}) + ((0.125 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.083917 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.083917 \text{ kipft/ft})}{(0.019904 \text{ kip/ft})}$$

$$E = 4.216 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.083917 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.019904 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.083917 \text{ kipft/ft})) + (4 \times (0.019904 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0728 \text{ ft}$$

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

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

$$V_{max} = ((0.019904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0728 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0728 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.12293 \text{ kip}$$

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

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

$$M_{max} = ((0.019904 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(4.216 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.4621 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.858 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.401 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.401 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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 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{(5.858 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0021898</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.858 \text{ kip} \rightarrow 5858 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5858 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.27 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.27 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.27 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.27 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.6 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 17.668 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(17.668 \text{ kip})}{(110.6 \text{ kip})}$$

$$Ratio = 0.15974$$

Status: **PASS**  
Ratio: **0.160**

**Considering z-direction:**

$V_{max} = 0.12293 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.12293 \text{ kip})}{(110.6 \text{ kip})}$$

$$Ratio = 0.0011115$$

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  $\phi M_{n,2}$

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

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

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

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:**

$M_{max} = 71.552 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(71.552 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.28666$$

Status: **PASS**  
Ratio: **0.290**

**Considering z-direction:**

$M_{max} = 0.4621 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.4621 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0018514$$

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

REFERENCES	CALCULATIONS	RESULTS
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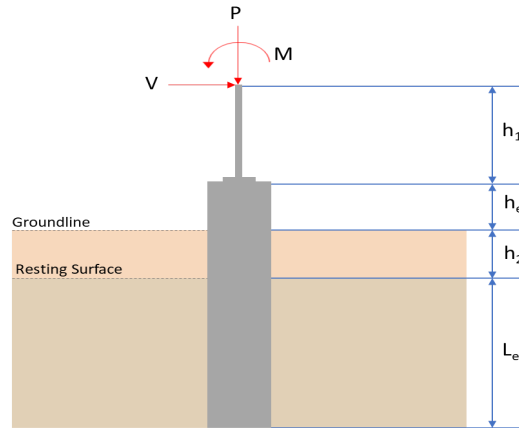
## SkyCiv Foundation Design

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

$L = 8.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.278	5.858
$V_x$ (kip)	-4.314	-7.189
$V_z$ (kip)	-0.084	-0.125
$M_x$ (kipft)	-0.355	-0.527
$M_z$ (kipft)	65.713	110.462

### 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,

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

$$H_o = \frac{(-4.314 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68694 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(65.713 \text{ kipft}) + ((-4.314 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 10.464 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.9801 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.084 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.013376 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.355 \text{ kipft}) + ((-0.084 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.056529 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.4924 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.9801 \text{ ft}), (1.4924 \text{ ft})]$$

$$L_{e,req} = 7.98 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.98 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.93882$$

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

### 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

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

$$q = \frac{(4.278 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.26737 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.26737 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.13369$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.68694 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 10.464 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (10.464 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (10.464 \text{ kipft/ft})) + (4 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8587 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (10.464 \text{ kipft/ft})) + (3 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (10.464 \text{ kipft/ft})) + (2 \times (-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.31192 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (10.464 \text{ kipft/ft})) + ((-0.68694 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.253 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8587 \text{ ft})}{2}$$

$$p_a = 0.4394 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.31192 \text{ kip/ft}^2)}{(0.4394 \text{ kip/ft}^2)}$$

$$Ratio = 0.70987$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.253 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.98278$$

Status: **PASS**  
Ratio: **0.710**

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

#### Considering z-direction:

$H_o = -0.013376 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.056529 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.056529 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.013376 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.056529 \text{ kipft/ft})) + (4 \times (-0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0724 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.056529 \text{ kipft/ft})) + (3 \times (-0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.056529 \text{ kipft/ft})) + (2 \times (-0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.0023737 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.056529 \text{ kipft/ft})) + ((-0.013376 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.000052895 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0724 \text{ ft})}{2}$$

$$p_a = 0.45543 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0023737 \text{ kip/ft}^2)}{(0.45543 \text{ kip/ft}^2)}$$

$$Ratio = -0.0052121$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

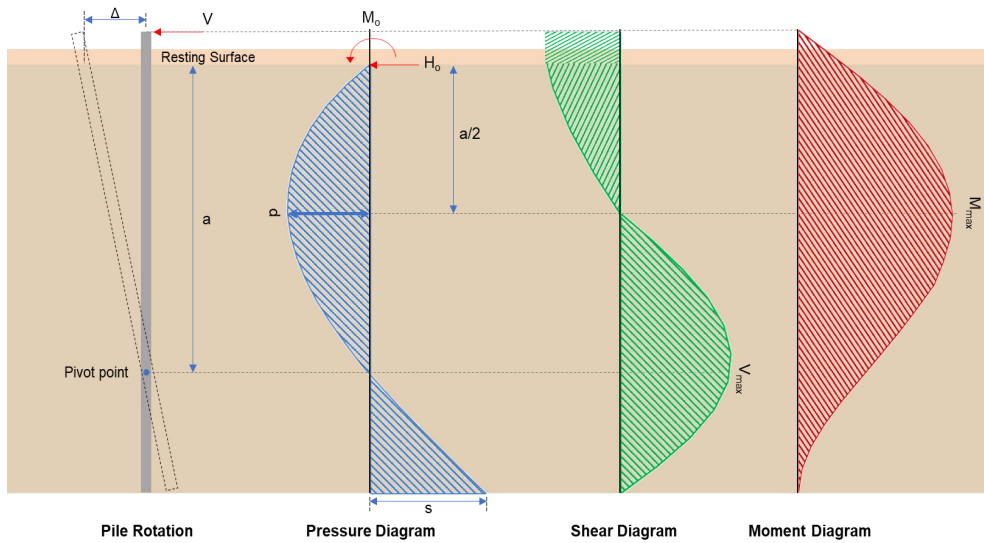
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.000052895 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = -0.000041486$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-7.189 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1447 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(110.46 \text{ kipft}) + ((-7.189 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 17.589 \text{ kipft/ft}$$

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

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

$$E = \frac{(17.589 \text{ kipft/ft})}{(-1.1447 \text{ kip/ft})}$$

$$E = 15.365 \text{ ft}$$

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

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

$$a = \frac{(4 \times (17.589 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (17.589 \text{ kipft/ft})) + (4 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = \frac{(6 \times (17.589 \text{ kipft/ft})) + (4 \times (-1.1447 \text{ kip/ft}) \times (8.5 \text{ ft}))}{}$$

$$a = 5.8575 \text{ ft}$$

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

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

$$V_{max} = ((-1.1447 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.8575 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.8575 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.668 \text{ kip}$$

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

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

$$M_{max} = ((-1.1447 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(15.365 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8575 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.8575 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (15.365 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.8575 \text{ ft})}{(2 \times (8.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 71.552 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.125 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.019904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.527 \text{ kipft}) + ((-0.125 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.083917 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.083917 \text{ kipft/ft})}{(-0.019904 \text{ kip/ft})}$$

$$E = 4.216 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.083917 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.019904 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.083917 \text{ kipft/ft})) + (4 \times (-0.019904 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0728 \text{ ft}$$

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

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

$$V_{max} = ((-0.019904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0728 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0728 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.12293 \text{ kip}$$

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

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

$$M_{max} = ((-0.019904 \text{ kip/ft}) \times (48 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(4.216 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.216 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0728 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.4621 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.858 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.401 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.401 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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. 10: 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{(5.858 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0021898</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.858 \text{ kip} \rightarrow 5858 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(5858 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.27 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.27 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.27 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.27 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.6 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 17.668 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(17.668 \text{ kip})}{(110.6 \text{ kip})}$$

$$Ratio = 0.15974$$

Status: **PASS**  
Ratio: **0.160**

**Considering z-direction:**

$V_{max} = 0.12293 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.12293 \text{ kip})}{(110.6 \text{ kip})}$$

$$Ratio = 0.0011115$$

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:**

$M_{max} = 71.552 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(71.552 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.28667$$

Status: **PASS**  
Ratio: **0.290**

**Considering z-direction:**

$M_{max} = 0.4621 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.4621 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0018514$$

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

REFERENCES	CALCULATIONS	RESULTS
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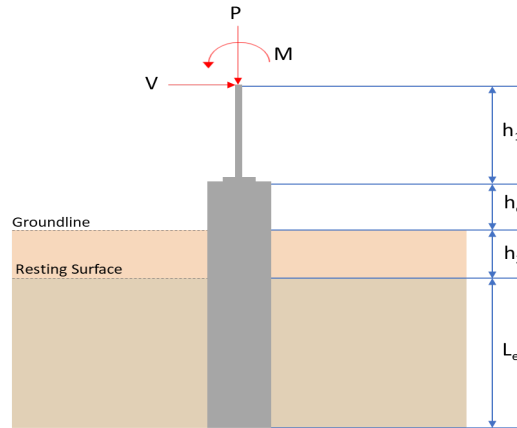
## SkyCiv Foundation Design

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

$L = 9$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.780	6.569
$V_x$ (kip)	-4.869	-8.111
$V_z$ (kip)	-0.009	-0.014
$M_x$ (kipft)	-0.033	-0.055
$M_z$ (kipft)	74.139	124.657

### 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,

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

$$H_o = \frac{(-4.869 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77532 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(74.139 \text{ kipft}) + ((-4.869 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.806 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2466 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.009 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0014331 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((-0.009 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0052548 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.71073 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(8.2466 \text{ ft}), (0.71073 \text{ ft})]$$

$$L_{e,req} = 8.247 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(8.247 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.91633$$

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

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

$$q = \frac{(4.78 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.29875 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.29875 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14938$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.77532 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 11.806 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (11.806 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (11.806 \text{ kipft/ft})) + (4 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.212 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.806 \text{ kipft/ft})) + (3 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (11.806 \text{ kipft/ft})) + (2 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.29817 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (11.806 \text{ kipft/ft})) + ((-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2321 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.212 \text{ ft})}{2}$$

$$p_a = 0.4659 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.29817 \text{ kip/ft}^2)}{(0.4659 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2321 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91266$$

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

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

#### Considering z-direction:

$H_o = -0.0014331 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.0052548 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.0052548 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0014331 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.0052548 \text{ kipft/ft})) + (4 \times (-0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4655 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0052548 \text{ kipft/ft})) + (3 \times (-0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.0052548 \text{ kipft/ft})) + (2 \times (-0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = -0.00028835 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.0052548 \text{ kipft/ft})) + ((-0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = -0.00017693 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4655 \text{ ft})}{2}$$

$$p_a = 0.48491 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00028835 \text{ kip/ft}^2)}{(0.48491 \text{ kip/ft}^2)}$$

$$Ratio = -0.00059464$$

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

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

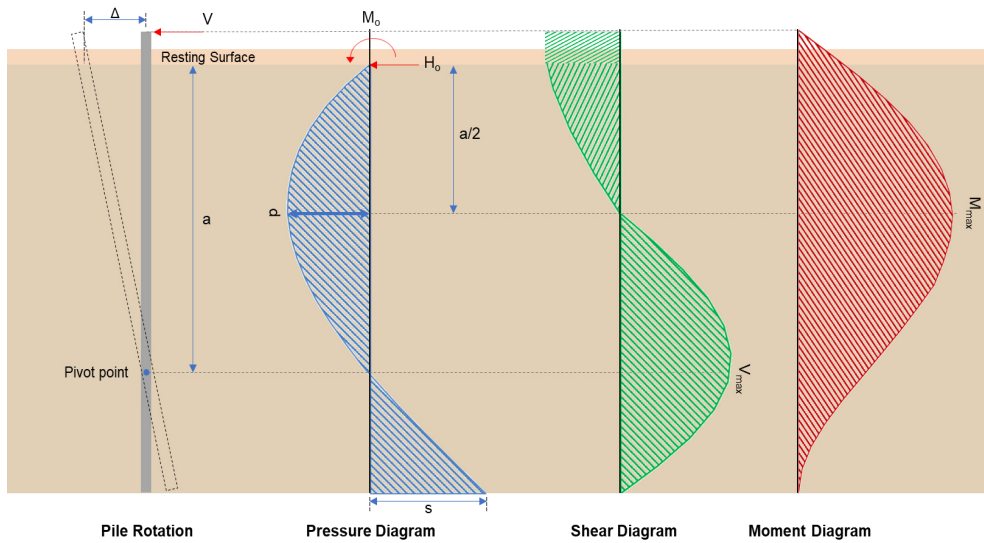
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00017693 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = -0.00013106$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-8.111 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2916 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(124.66 \text{ kipft}) + ((-8.111 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 19.85 \text{ kipft/ft}$$

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

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

$$E = \frac{(19.85 \text{ kipft/ft})}{(-1.2916 \text{ kip/ft})}$$

$$E = 15.369 \text{ ft}$$

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

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

$$a = \frac{(4 \times (19.85 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.2916 \text{ kip/ft}) \times (9 \text{ ft})^2)}{6 \times (19.85 \text{ kipft/ft}) + 4 \times (-1.2916 \text{ kip/ft}) \times 9 \text{ ft}}$$

$$a = \frac{(6 \times (19.85 \text{ kip/ft})) + (4 \times (-1.2916 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.2106 \text{ ft}$$

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

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

$$V_{max} = ((-1.2916 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2106 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2106 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 19.018 \text{ kip}$$

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

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

$$M_{max} = ((-1.2916 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[ \left( \frac{(15.369 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 81.361 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

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

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

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.008758 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.008758 \text{ kipft/ft})}{(-0.0022293 \text{ kip/ft})}$$

$$E = 3.9286 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4532 \text{ ft}$$

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

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

$$V_{max} = ((-0.0022293 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.4532 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.4532 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.012841 \text{ kip}$$

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

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

$$M_{max} = ((-0.0022293 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[ \left( \frac{(3.9286 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050641 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(0.569 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.378 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.378 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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 <math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> <p>25.7.2.2 Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p>25.7.2.1 <math>s_{ties}</math> - Maximum spacing of ties,</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p style="text-align: center;"><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p style="text-align: center;"><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 style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>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 <math>\phi P_N</math> - Allowable axial compressive strength</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y 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><i>Ratio - Capacity</i></p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(6.569 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0024555</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2 <math>b_w</math> = 48 in - Effective width, <math>d</math> - Effective depth</p> <p>22.5.5.1.3 <math>\lambda_s</math> - size effect modification factor</p> <p>22.5.5.1.1 <math>V_{c,max}</math> - Max shear strength of concrete</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></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 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 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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 6.569 \text{ kip} \rightarrow 6569 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.36 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.36 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.36 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.36 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.67 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 19.018 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.018 \text{ kip})}{(110.67 \text{ kip})}$$

$$Ratio = 0.17185$$

**Considering z-direction:**

$V_{max} = 0.012841 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.012841 \text{ kip})}{(110.67 \text{ kip})}$$

$$Ratio = 0.00011604$$

Status: **PASS**  
Ratio: **0.170**

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

$\phi M_{n,2}$

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

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

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

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:**

$M_{max} = 81.361 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(81.361 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.32597$$

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

**Considering z-direction:**

$M_{max} = 0.050641 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$\text{Ratio} = \frac{(0.050641 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00020289$$

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

REFERENCES	CALCULATIONS	RESULTS
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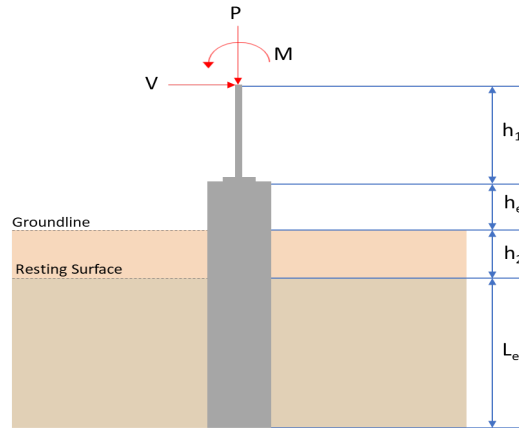
## SkyCiv Foundation Design

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

$L = 9$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.786	6.579
$V_x$ (kip)	-4.913	-8.188
$V_z$ (kip)	0.000	0.000
$M_x$ (kipft)	0.000	0.000
$M_z$ (kipft)	74.954	126.087

### 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,

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

$$H_o = \frac{(-4.913 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.78232 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(74.954 \text{ kipft}) + ((-4.913 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.935 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2741 \text{ ft}$  - 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,

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

$$L_{e,req} = \text{MAX}[(8.2741 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 8.274 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

*Ratio* - Embedded depth

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

$$\text{Ratio} = \frac{(8.274 \text{ ft})}{(9 \text{ ft})}$$

$$\text{Ratio} = 0.91933$$

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

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

$$q = \frac{(4.786 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.29913 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(0.29913 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14956$$

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

Czerniak

### Lateral Soil Pressure (ASD):

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

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.78232 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 11.935 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (11.935 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.78232 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (11.935 \text{ kipft/ft})) + (4 \times (-0.78232 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2117 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.935 \text{ kipft/ft})) + (3 \times (-0.78232 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (11.935 \text{ kipft/ft})) + (2 \times (-0.78232 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.302 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (11.935 \text{ kipft/ft})) + ((-0.78232 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2467 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2117 \text{ ft})}{2}$$

$$p_a = 0.46588 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.302 \text{ kip/ft}^2)}{(0.46588 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64823$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

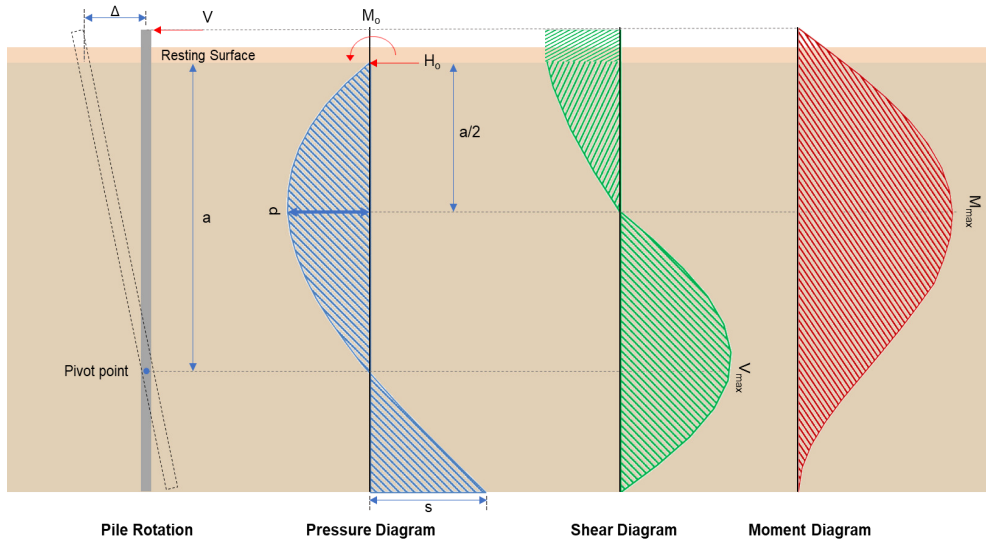
$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2467 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

Status: **PASS**  
Ratio: **0.650**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-8.188 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3038 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(126.09 \text{ kipft}) + ((-8.188 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.078 \text{ kipft/ft}$$

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

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

$$E = \frac{(20.078 \text{ kipft/ft})}{(-1.3038 \text{ kip/ft})}$$

$$E = 15.399 \text{ ft}$$

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

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

$$a = \frac{(4 \times (20.078 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3038 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (20.078 \text{ kipft/ft})) + (4 \times (-1.3038 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2103 \text{ ft}$$

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

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

$$V_{max} = ((-1.3038 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.399 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2103 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.399 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2103 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$v_{max} = 19.23 \text{ kip}$$

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

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

$$M_{max} = ((-1.3038 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[ \left( \frac{(15.399 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2103 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.399 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2103 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (15.399 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2103 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 82.272 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\left( \frac{6.579 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.378 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.378 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

$$Ratio = \frac{(6.579 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0024593$$

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

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

**Parameters:**

$b_w = 48 \text{ in}$  - Effective width,

22.5.2.2  $d$  - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 6.579 \text{ kip} \rightarrow 6579 \text{ lbf}$ ,

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

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(6579 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.36 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.36 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.36 \text{ kip}$$

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)

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

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

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

$V_s$  - 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

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

$$\phi V_n = (0.65) \times ((119.36 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.67 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 19.23 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(19.23 \text{ kip})}{(110.67 \text{ kip})}$$

$$\text{Ratio} = 0.17376$$

Status: **PASS**  
 Ratio: **0.170**

**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:** $M_{max} = 82.272 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(82.272 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.32962$$

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

REFERENCES	CALCULATIONS	RESULTS
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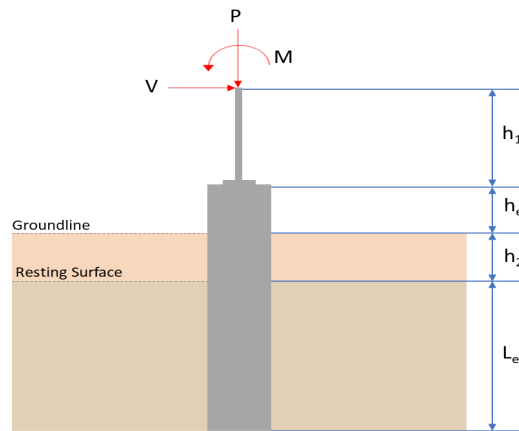
## SkyCiv Foundation Design

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

$L = 9$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	4.780	6.569
$V_x$ (kip)	-4.869	-8.111
$V_z$ (kip)	0.009	0.014
$M_x$ (kipft)	0.033	0.055
$M_z$ (kipft)	74.139	124.657

### 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,

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

$$H_o = \frac{(-4.869 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77532 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(74.139 \text{ kipft}) + ((-4.869 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.806 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2466 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.009 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0014331 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.033 \text{ kipft}) + ((0.009 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0052548 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.78705 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(8.2466 \text{ ft}), (0.78705 \text{ ft})]$$

$$L_{e,req} = 8.247 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(8.247 \text{ ft})}{(9 \text{ ft})}$$

$$Ratio = 0.91633$$

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

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

$$q = \frac{(4.78 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.29875 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.29875 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14938$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(9 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.77532 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 11.806 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (11.806 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (11.806 \text{ kipft/ft})) + (4 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.212 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.806 \text{ kipft/ft})) + (3 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (11.806 \text{ kipft/ft})) + (2 \times (-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.29817 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (11.806 \text{ kipft/ft})) + ((-0.77532 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2321 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.212 \text{ ft})}{2}$$

$$p_a = 0.4659 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.29817 \text{ kip/ft}^2)}{(0.4659 \text{ kip/ft}^2)}$$

$$Ratio = 0.64$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2321 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.91266$$

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

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

#### Considering z-direction:

$H_o = 0.0014331 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.0052548 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.0052548 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0014331 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.0052548 \text{ kipft/ft})) + (4 \times (0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4655 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0052548 \text{ kipft/ft})) + (3 \times (0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (0.0052548 \text{ kipft/ft})) + (2 \times (0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.0007944 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.0052548 \text{ kipft/ft})) + ((0.0014331 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 0.0017339 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.4655 \text{ ft})}{2}$$

$$p_a = 0.48491 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0007944 \text{ kip/ft}^2)}{(0.48491 \text{ kip/ft}^2)}$$

$$Ratio = 0.0016382$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

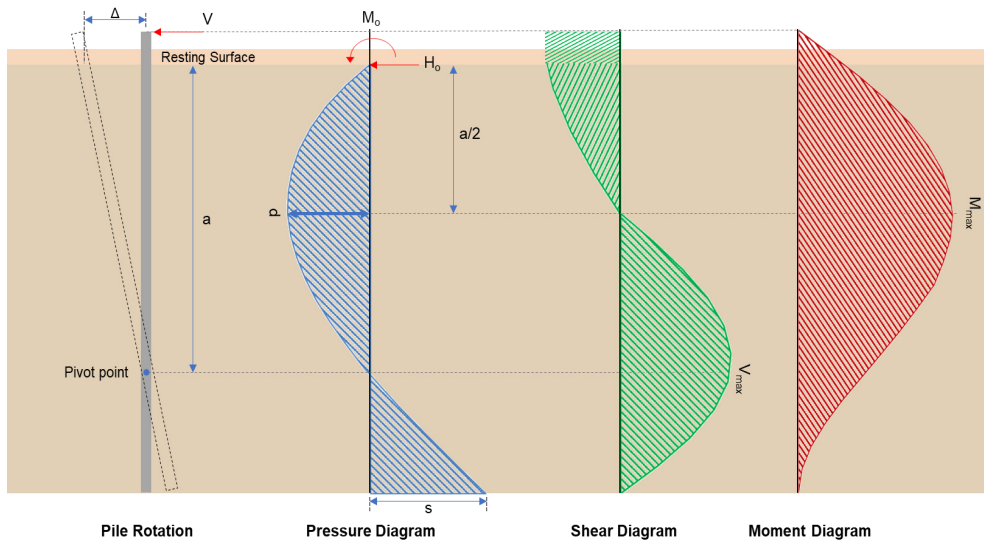
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0017339 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$Ratio = 0.0012844$$

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,

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

$$H_o = \frac{(-8.111 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2916 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(124.66 \text{ kipft}) + ((-8.111 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 19.85 \text{ kipft/ft}$$

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

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

$$E = \frac{(19.85 \text{ kipft/ft})}{(-1.2916 \text{ kip/ft})}$$

$$E = 15.369 \text{ ft}$$

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

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

$$a = \frac{(4 \times (19.85 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.2916 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (19.85 \text{ kipft/ft})) + (4 \times (-1.2916 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = \frac{(6 \times (19.85 \text{ kip/ft})) + (4 \times (-1.2916 \text{ kip/ft}) \times (9 \text{ ft}))}{}$$

$$a = 6.2106 \text{ ft}$$

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

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

$$V_{max} = ((-1.2916 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2106 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2106 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 19.018 \text{ kip}$$

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

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

$$M_{max} = ((-1.2916 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[ \left( \frac{(15.369 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.369 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.2106 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 81.361 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

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

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

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.055 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.008758 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.008758 \text{ kipft/ft})}{(0.0022293 \text{ kip/ft})}$$

$$E = 3.9286 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.008758 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (0.008758 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.4532 \text{ ft}$$

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

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

$$V_{max} = ((0.0022293 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.4532 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.4532 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.012841 \text{ kip}$$

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

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

$$M_{max} = ((0.0022293 \text{ kip/ft}) \times (48 \text{ in}) \times (9 \text{ ft})) \times \left[ \left( \frac{(3.9286 \text{ ft})}{(9 \text{ ft})} + \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left( \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.9286 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left( \frac{(6.4532 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050641 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(0.569 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.378 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.378 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(6.569 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0024555</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 6.569 \text{ kip} \rightarrow 6569 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.36 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.36 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.36 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.36 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.67 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 19.018 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.018 \text{ kip})}{(110.67 \text{ kip})}$$

$$Ratio = 0.17185$$

**Considering z-direction:**

$V_{max} = 0.012841 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.012841 \text{ kip})}{(110.67 \text{ kip})}$$

$$Ratio = 0.00011604$$

Status: **PASS**  
Ratio: **0.170**

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

$\phi M_{n,2}$

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

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

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

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:**

$M_{max} = 81.361 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(81.361 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.32597$$

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

**Considering z-direction:**

$M_{max} = 0.050641 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$\text{Ratio} = \frac{(0.050641 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00020289$$

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