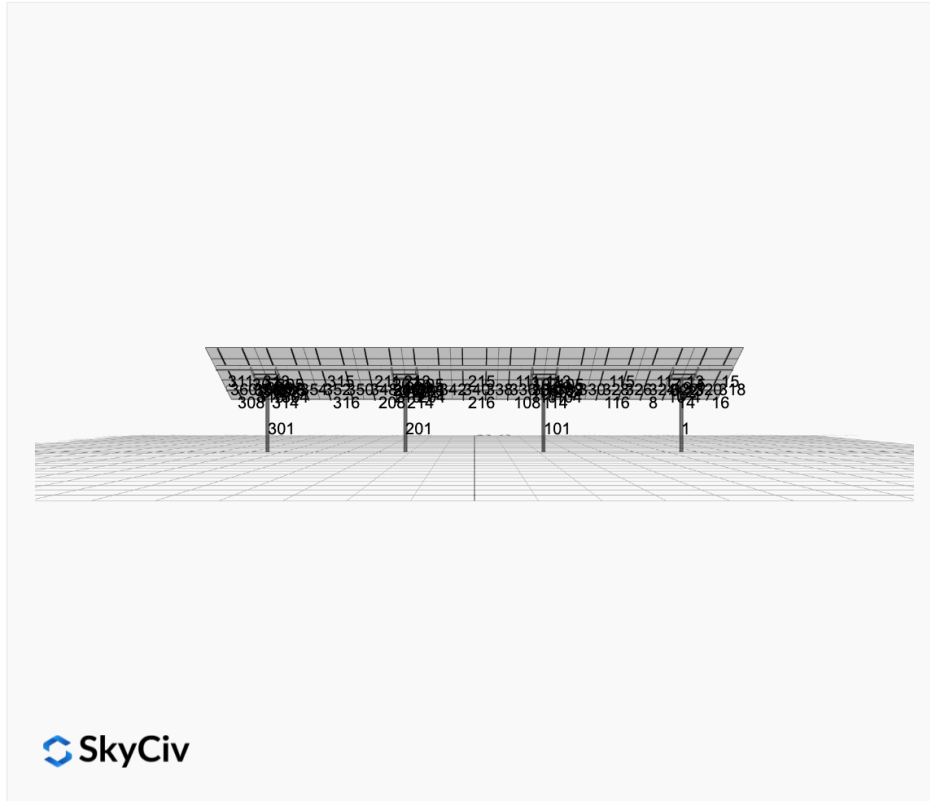


# Project Details



**Project Name:** Carnegie Building - v3cu      **Date:** Wed Feb 26 2025  
**Location:** 412 W Miner St, Yreka, CA 96097, USA      **Number of Modules:** 55  
**Unique ID:** 4P-22.5-6TOP-XD-45-L-5Hx11W-2EIF      **Number of Poles:** 4  
**Dealer:** \_\_\_\_\_      **Date Sold:** \_\_\_\_\_



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	83.42 ft
Winter Tilt Angle	25
Front Edge Clearance	8 ft

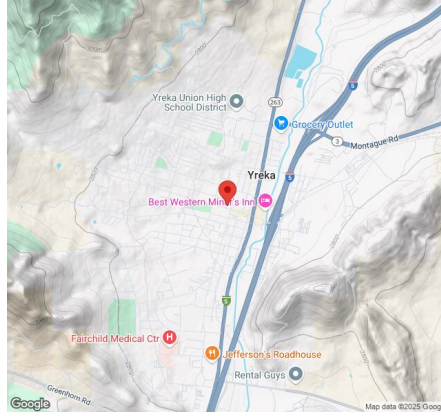
## MT Solar Bill of Materials (4P-22.5-6TOP-XD-45-L-5Hx11W-2EIF)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	12
MTS-CLAMP-ANGLE-4PK	Angle Clamp	11

## Rail Bill of Materials

Part	Qty
Rails (223in)	22
Rail Attachment	88
Module Mid Clamp	88
Module End Clamp	44
Ground Lug	11

## Site Details:



**Site Address:** 412 W Miner St, Yreka, CA 96097, USA

### Array Specification

<b>Duty Classification:</b>	XD
<b>Module Width:</b>	44.60 in
<b>Module Length:</b>	90.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	11
<b>Total Number of Modules:</b>	55
<b>Winter Tilt Angle:</b>	25
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	15.94 ft
<b>Total Frame Length:</b>	82.50 ft
<b>Frame Weight:</b>	5144 lbs
<b>Array Dimensions N/S:</b>	18.79 ft
<b>Array Dimensions E/W:</b>	83.42 ft
<b>Rail Length:</b>	225.50 in
<b>Rail Spacing:</b>	3.79 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	11.97 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	22.5 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.75 ft Pile 2: 6.00 ft Pile 3: 6.00 ft Pile 4: 5.75 ft
<b>Foundation Volume:</b>	13.926 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	412 W Miner St, Yreka, CA 96097, USA
<b>Wind Speed:</b>	90 mph

**Snow Load:**

40 psf

### **Design Disclaimer**

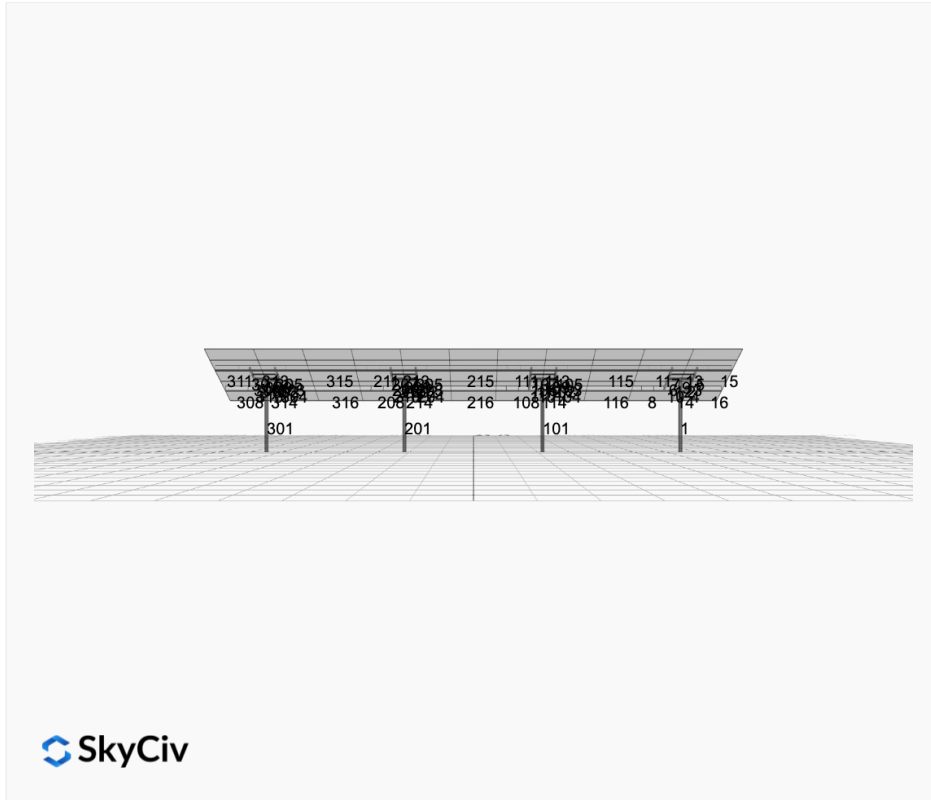
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

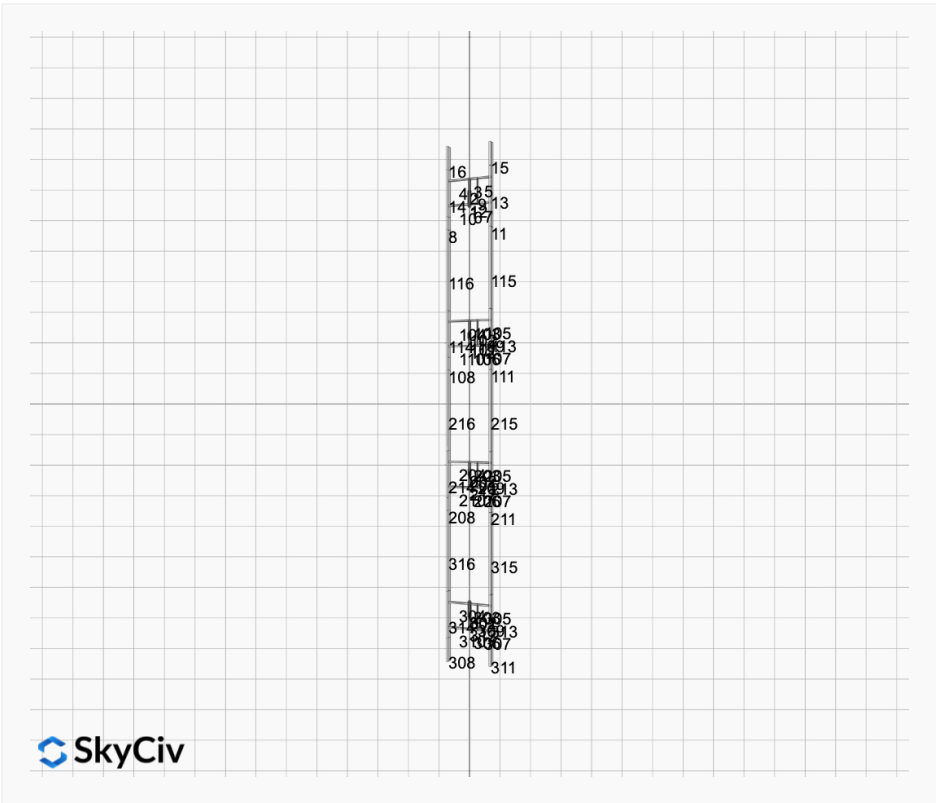
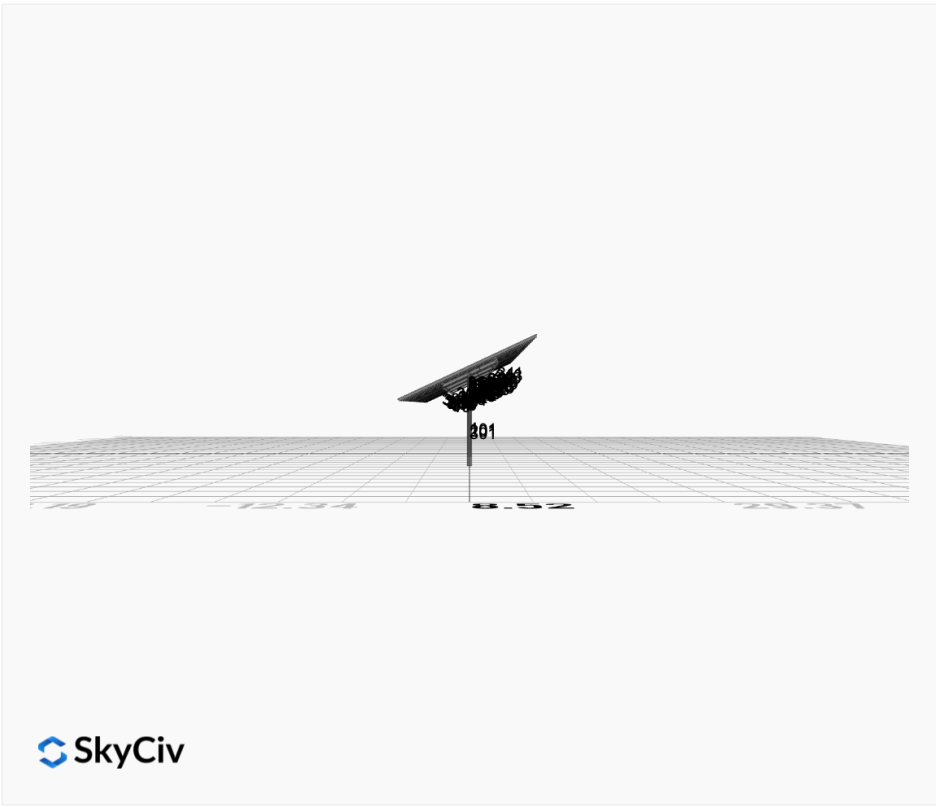
## AutoDesigner Input

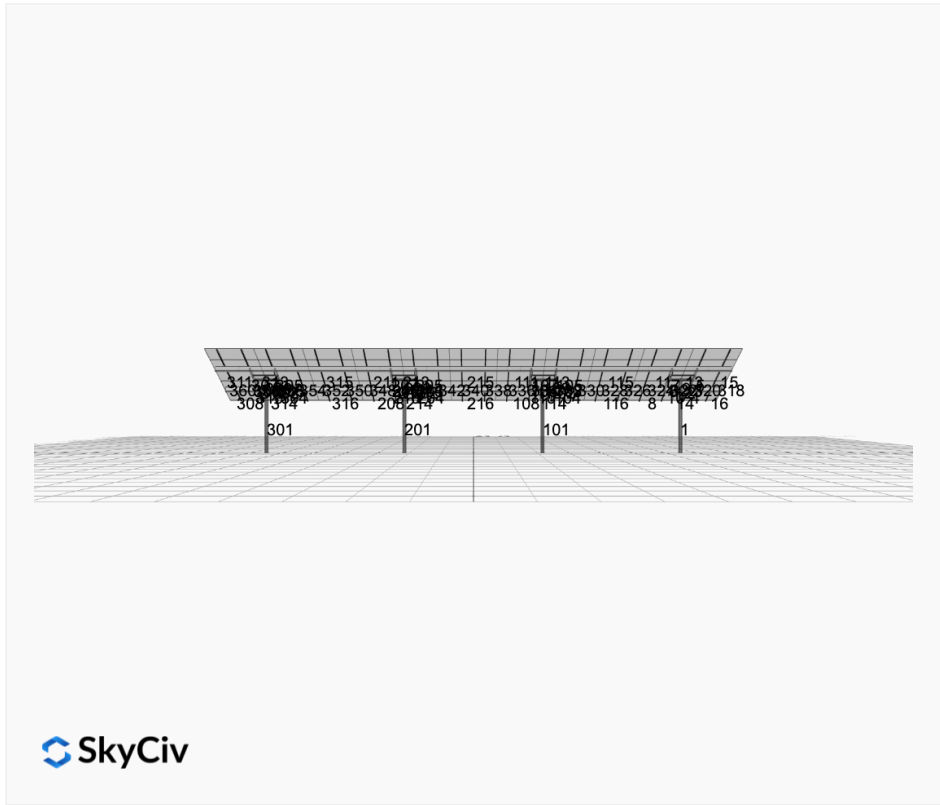
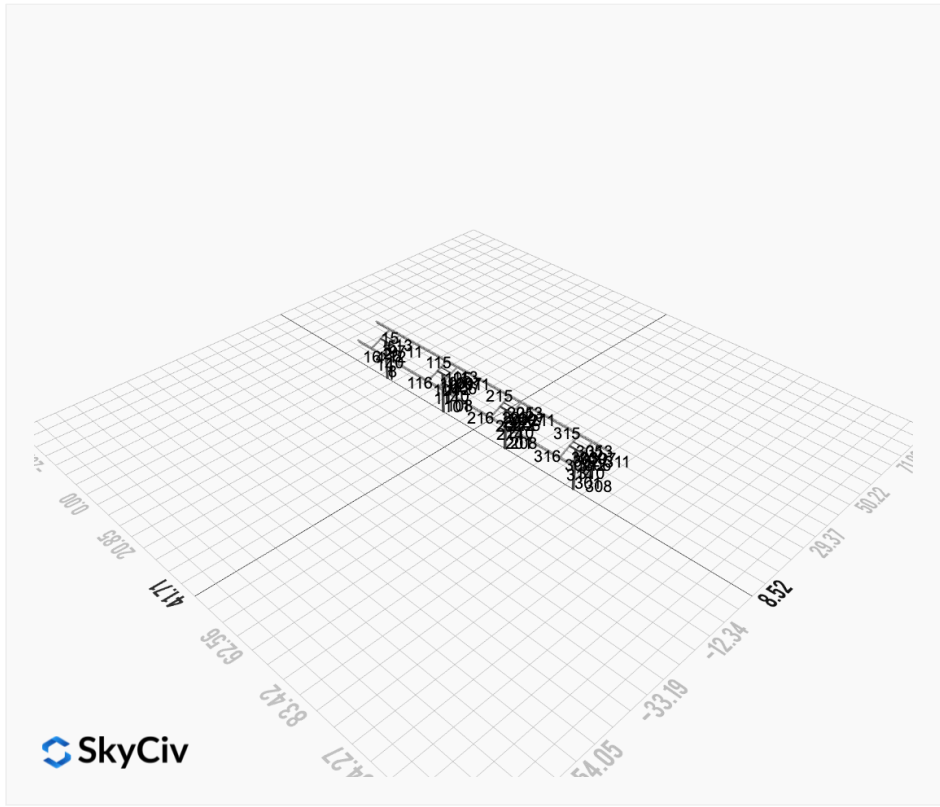
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### Design Notes:

- 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)

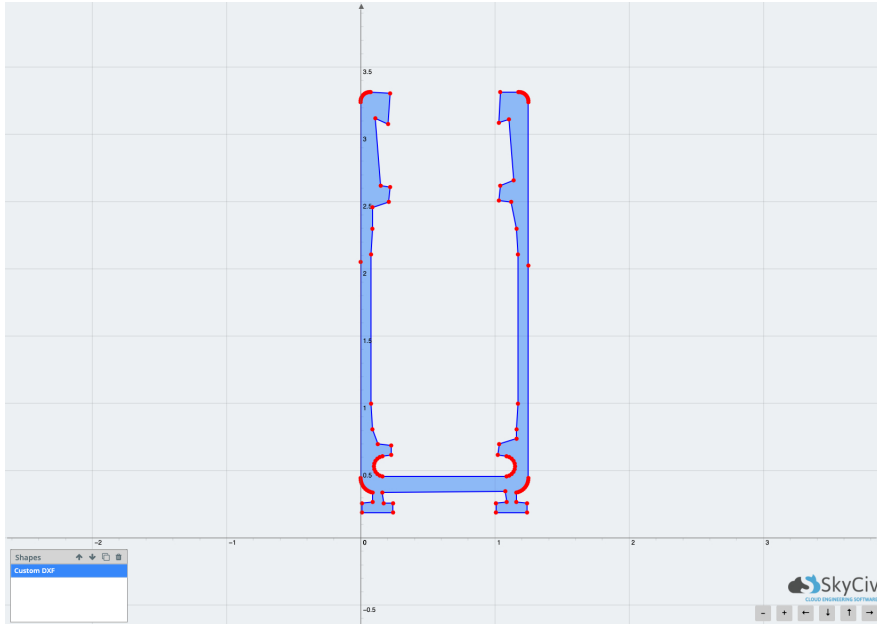






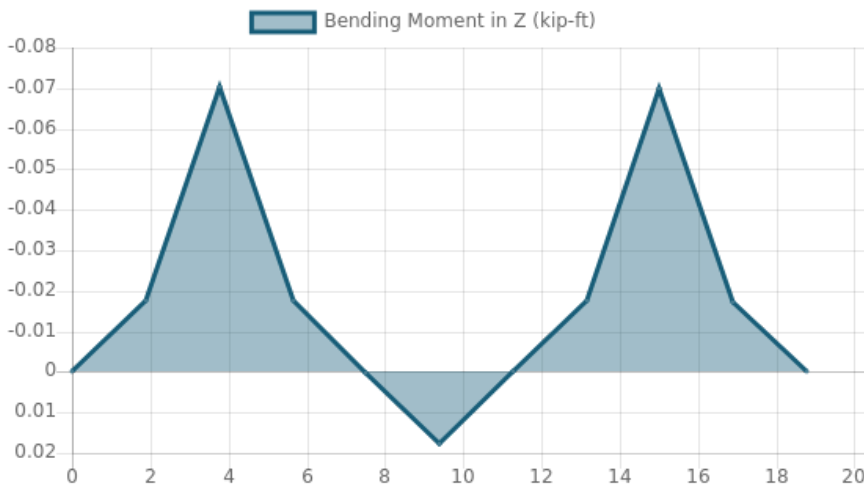
### Rail Design Check

**Rail Length:** 18.79166666666668 ft  
**Additional Restraints Required:** 4ft Spread Clamps  
**Tributary Width:** 3.791666666666665 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Snow (X):** 0.0680 kip/ft  
**Snow (Y):** -0.0317 kip/ft  
**Wind uplift Case A:** 0.0495 kip/ft  
**Wind uplift Case A:** 0.0495 kip/ft  
**Wind uplift Case B (X):** 0.0000 kip/ft  
**Wind uplift Case B (Y):** 0.0722 kip/ft

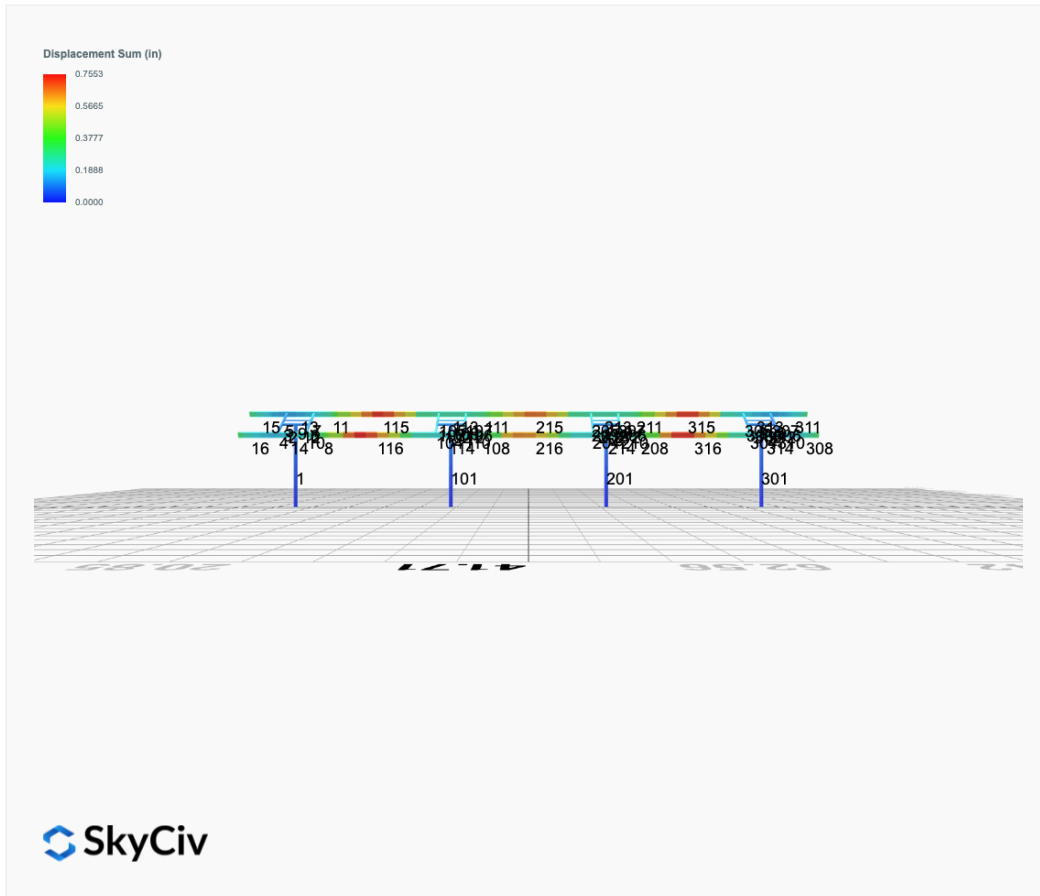


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	13.70606065	0.397	PASS
Material Yield	34.5	13.70606065	0.397	PASS
Material Strength	37	13.70606065	0.370	PASS

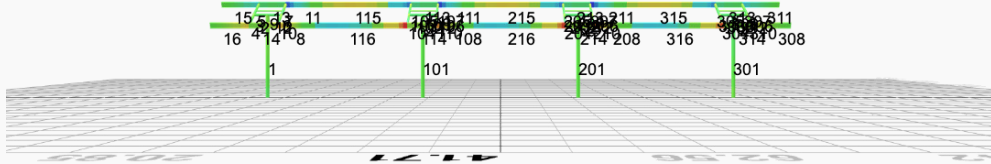
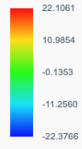
Member 1, ULS: 1. 1.4D



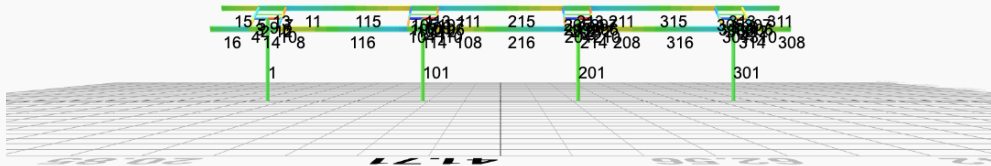
# FEM Results (Envelope Worst Case for each member)



Top Bending Stress Y (ksi)



Shear Stress Y (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	0.0193	2.4729	0.0546	0.2110	-0.0271	-0.1711
ULS: 2. D + L	0.0193	2.4729	0.0546	0.2110	-0.0271	-0.1711
ULS: 3. D + (S or Lr or R)	0.0788	8.6319	0.2264	0.8784	-0.1135	-0.7819
ULS: 3. D + (S or Lr or R)	0.0193	2.4729	0.0546	0.2110	-0.0271	-0.1711
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0639	7.0921	0.1835	0.7116	-0.0919	-0.6292
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0193	2.4729	0.0546	0.2110	-0.0271	-0.1711
ULS: 5b. D + 0.7E	0.0193	2.4729	0.0546	0.2110	-0.0271	-0.1711
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0639	7.0921	0.1835	0.7116	-0.0919	-0.6292
ULS: 8. 0.6D + 0.7E	0.0116	1.4837	0.0327	0.1266	-0.0163	-0.1027
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3044	5.2367	0.1730	0.6636	-0.1839	16.7799
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3044	5.2367	0.1730	0.6636	-0.1839	16.7799
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1706	0.0591	-0.0419	-0.1553	0.0996	-13.4613
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0284	0.3905	-0.0492	-0.1827	0.1130	-18.2806
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9289	9.1650	0.2723	1.0510	-0.2095	12.0841
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9289	9.1650	0.2723	1.0510	-0.2095	12.0841
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9274	5.2818	0.1111	0.4368	0.0032	-10.5968
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8208	5.5303	0.1056	0.4163	0.0132	-14.2113
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9735	4.5457	0.1434	0.5504	-0.1447	12.5422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9735	4.5457	0.1434	0.5504	-0.1447	12.5422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8827	0.6626	-0.0178	-0.0638	0.0679	-10.1387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7762	0.9111	-0.0232	-0.0843	0.0780	-13.7532
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3121	4.2475	0.1512	0.5792	-0.1731	16.8484
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3121	4.2475	0.1512	0.5792	-0.1731	16.8484
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1628	-0.9300	-0.0638	-0.2398	0.1105	-13.3928
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0207	-0.5987	-0.0710	-0.2671	0.1239	-18.2122

### 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	15.1318
Shear X	-2.2062
Shear Z	0.4520
Moment X	1.7617
Moment Y (Twist)	0.3508
Moment Z	32.7236

### 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	9.1650
Shear X	-1.3121
Shear Z	0.2723
Moment X	1.0510
Moment Y (Twist)	0.2095
Moment Z	18.2806

## 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	-0.0193	2.9820	-0.0046	-0.0178	0.0044	0.2361
ULS: 2. D + L	-0.0193	2.9820	-0.0046	-0.0178	0.0044	0.2361
ULS: 3. D + (S or Lr or R)	-0.0788	10.7285	-0.0185	-0.0723	0.0175	0.9400
ULS: 3. D + (S or Lr or R)	-0.0193	2.9820	-0.0046	-0.0178	0.0044	0.2361
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0639	8.7919	-0.0150	-0.0587	0.0142	0.7640

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0193	2.9820	-0.0046	-0.0178	0.0044	0.2361
ULS: 5b. D + 0.7E	-0.0193	2.9820	-0.0046	-0.0178	0.0044	0.2361
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0639	8.7919	-0.0150	-0.0587	0.0142	0.7640
ULS: 8. 0.6D + 0.7E	-0.0116	1.7892	-0.0028	-0.0107	0.0026	0.1417
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5998	6.4464	0.0073	0.0265	-0.0223	20.1637
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5998	6.4464	0.0073	0.0265	-0.0223	20.1637
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3707	-0.0540	-0.0109	-0.0409	0.0211	-15.3377
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1498	0.3932	-0.0216	-0.0814	0.0382	-20.4875
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2493	11.3902	-0.0061	-0.0255	-0.0058	15.7097
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2493	11.3902	-0.0061	-0.0255	-0.0058	15.7097
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9786	6.5149	-0.0197	-0.0760	0.0267	-10.9163
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8129	6.8503	-0.0277	-0.1063	0.0395	-14.7787
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2047	5.5803	0.0043	0.0154	-0.0156	15.1818
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2047	5.5803	0.0043	0.0154	-0.0156	15.1818
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0232	0.7050	-0.0093	-0.0351	0.0169	-11.4442
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8575	1.0404	-0.0173	-0.0655	0.0297	-15.3066
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5921	5.2536	0.0091	0.0336	-0.0240	20.0692
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5921	5.2536	0.0091	0.0336	-0.0240	20.0692
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3784	-1.2468	-0.0090	-0.0338	0.0193	-15.4321
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1575	-0.7996	-0.0197	-0.0742	0.0364	-20.5819

### 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	18.8530
Shear X	-2.6774
Shear Z	-0.0485
Moment X	-0.1882
Moment Y (Twist)	0.0773
Moment Z	36.4004

### 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	11.3902
Shear X	-1.5998
Shear Z	-0.0277
Moment X	-0.1063
Moment Y (Twist)	0.0395
Moment Z	20.5819

### 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	-0.0193	2.9820	0.0046	0.0178	-0.0044	0.2361
ULS: 2. D + L	-0.0193	2.9820	0.0046	0.0178	-0.0044	0.2361
ULS: 3. D + (S or Lr or R)	-0.0788	10.7285	0.0185	0.0722	-0.0172	0.9400
ULS: 3. D + (S or Lr or R)	-0.0193	2.9820	0.0046	0.0178	-0.0044	0.2361
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0639	8.7919	0.0150	0.0586	-0.0140	0.7640
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0193	2.9820	0.0046	0.0178	-0.0044	0.2361
ULS: 5b. D + 0.7E	-0.0193	2.9820	0.0046	0.0178	-0.0044	0.2361
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0639	8.7919	0.0150	0.0586	-0.0140	0.7640
ULS: 8. 0.6D + 0.7E	-0.0116	1.7892	0.0028	0.0107	-0.0026	0.1417
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5998	6.4464	-0.0073	-0.0265	0.0223	20.1637
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5998	6.4464	-0.0073	-0.0265	0.0223	20.1637
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3707	-0.0540	0.0109	0.0409	-0.0210	-15.3377
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1498	0.3932	0.0216	0.0814	-0.0381	-20.4875

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2493	11.3902	0.0061	0.0254	0.0060	15.7097
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2493	11.3902	0.0061	0.0254	0.0060	15.7097
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9786	6.5149	0.0197	0.0759	-0.0265	-10.9163
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8129	6.8503	0.0277	0.1063	-0.0393	-14.7787
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2047	5.5803	-0.0043	-0.0154	0.0156	15.1818
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2047	5.5803	-0.0043	-0.0154	0.0156	15.1818
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0232	0.7050	0.0093	0.0351	-0.0169	-11.4442
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8575	1.0404	0.0173	0.0655	-0.0297	-15.3066
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5921	5.2536	-0.0091	-0.0336	0.0240	20.0692
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5921	5.2536	-0.0091	-0.0336	0.0240	20.0692
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3784	-1.2468	0.0090	0.0338	-0.0193	-15.4321
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1575	-0.7996	0.0197	0.0742	-0.0364	-20.5819

### 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	18.8530
Shear X	-2.6774
Shear Z	0.0486
Moment X	0.1882
Moment Y (Twist)	0.0771
Moment Z	36.4007

### 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	11.3902
Shear X	-1.5998
Shear Z	0.0277
Moment X	0.1063
Moment Y (Twist)	0.0393
Moment Z	20.5819

### 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	0.0193	2.4729	-0.0546	-0.2110	0.0272	-0.1711
ULS: 2. D + L	0.0193	2.4729	-0.0546	-0.2110	0.0272	-0.1711
ULS: 3. D + (S or Lr or R)	0.0788	8.6318	-0.2264	-0.8787	0.1138	-0.7817
ULS: 3. D + (S or Lr or R)	0.0193	2.4729	-0.0546	-0.2110	0.0272	-0.1711
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0639	7.0921	-0.1835	-0.7118	0.0921	-0.6290
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0193	2.4729	-0.0546	-0.2110	0.0272	-0.1711
ULS: 5b. D + 0.7E	0.0193	2.4729	-0.0546	-0.2110	0.0272	-0.1711
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0639	7.0921	-0.1835	-0.7118	0.0921	-0.6290
ULS: 8. 0.6D + 0.7E	0.0116	1.4837	-0.0327	-0.1266	0.0163	-0.1026
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3044	5.2367	-0.1730	-0.6636	0.1839	16.7799
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3044	5.2367	-0.1730	-0.6636	0.1839	16.7799
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1706	0.0591	0.0419	0.1553	-0.0996	-13.4613
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0284	0.3905	0.0492	0.1827	-0.1130	-18.2806
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9289	9.1650	-0.2723	-1.0512	0.2097	12.0842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9289	9.1650	-0.2723	-1.0512	0.2097	12.0842
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9273	5.2818	-0.1111	-0.4370	-0.0029	-10.5967
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8208	5.5303	-0.1057	-0.4165	-0.0130	-14.2112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9735	4.5457	-0.1434	-0.5505	0.1447	12.5422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9735	4.5457	-0.1434	-0.5505	0.1447	12.5422
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8827	0.6626	0.0178	0.0637	-0.0679	-10.1387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7762	0.9111	0.0232	0.0842	-0.0779	-13.7532

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3121	4.2475	-0.1512	-0.5792	0.1731	16.8484
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3121	4.2475	-0.1512	-0.5792	0.1731	16.8484
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1628	-0.9300	0.0638	0.2397	-0.1105	-13.3928
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0207	-0.5987	0.0710	0.2671	-0.1238	-18.2122

### 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	15.1318
Shear X	-2.2062
Shear Z	-0.4521
Moment X	-1.7628
Moment Y (Twist)	0.3512
Moment Z	32.7246

### 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	9.1650
Shear X	-1.3121
Shear Z	-0.2723
Moment X	-1.0512
Moment Y (Twist)	0.2097
Moment Z	18.2806

## Project Details

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

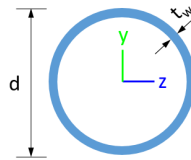


## Design Input Information

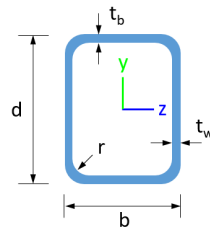
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

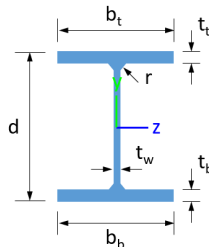
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	$t_w$ (in)	$t_b$ (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
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315	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.33,1.24,1.12,1.12,1.13,1.14,1.11,1.11,1.11,1.11,1.11,1.11,2.07,1.18,1.12,1.12,1.13,1.14	300	200	1
316	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.15,1.12,1.11,1.11,1.11,1.20,1.11,1.11,1.11,1.12,1.11,1.11,1.09,1.13,1.11,1.11,1.11,1.39	300	200	1

## Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	69.86	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	32.91	6.46	56.26	44.91
14	159.30	97.43	34.29	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	251.16	69.86	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.60	6.46	56.26	44.91
114	159.30	97.43	31.67	6.46	56.26	44.91
115	159.30	48.27	14.52	6.46	56.26	44.91
116	159.30	48.27	14.96	6.46	56.26	44.91
201	251.16	69.86	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30

212	251.01	240.88	27.10	27.10	75.30	75.30
213	159.30	97.43	31.59	6.46	56.26	44.91
214	159.30	97.43	31.67	6.46	56.26	44.91
215	159.30	48.27	13.40	6.46	56.26	44.91
216	159.30	48.27	14.79	6.46	56.26	44.91
301	251.16	69.86	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	55.15	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	55.15	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	32.90	6.46	56.26	44.91
314	159.30	97.43	34.31	6.46	56.26	44.91
315	159.30	48.27	14.76	6.46	56.26	44.91
316	159.30	48.27	14.48	6.46	56.26	44.91

## 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	0.217	0.774	0.086	0.029	0.006	0.809	#13	0.672	Not Required	Pass
2	0.004	0.386	0.085	0.087	0.015	0.464	#21	0.036	Not Required	Pass
3	0.007	0.584	0.042	0.057	0.010	0.600	#21	0.046	Not Required	Pass
4	0.006	0.579	0.130	0.058	0.029	0.678	#21	0.082	Not Required	Pass
5	0.007	0.362	0.120	0.058	0.032	0.394	#21	0.076	Not Required	Pass
6	0.011	0.718	0.088	0.072	0.017	0.811	#21	0.046	Not Required	Pass
7	0.011	0.446	0.231	0.071	0.059	0.502	#21	0.076	Not Required	Pass
8	0.003	0.106	0.285	0.045	0.024	0.289	#24	0.102	Not Required	Pass
9	0.019	0.064	0.077	0.002	0.004	0.150	#21	0.206	Not Required	Pass
10	0.011	0.682	0.214	0.068	0.045	0.798	#21	0.082	Not Required	Pass
11	0.006	0.097	0.296	0.048	0.024	0.306	#24	0.102	Not Required	Pass
12	0.003	0.520	0.098	0.110	0.017	0.609	#21	0.054	Not Required	Pass
13	0.008	0.199	0.619	0.059	0.029	0.732	#21	0.306	Not Required	Pass
14	0.009	0.187	0.605	0.057	0.029	0.718	#24	0.204	Not Required	Pass
15	0.000	0.056	0.154	0.025	0.012	0.210	#21	Not Required	Not Required	Pass
16	0.000	0.056	0.154	0.025	0.012	0.210	#21	Not Required	Not Required	Pass
101	0.270	0.861	0.009	0.036	0.001	0.951	#13	0.672	Not Required	Warn
102	0.006	0.572	0.116	0.124	0.017	0.685	#21	0.036	Not Required	Pass
103	0.011	0.806	0.055	0.080	0.001	0.867	#21	0.046	Not Required	Pass
104	0.011	0.815	0.235	0.081	0.049	0.964	#21	0.082	Not Required	Warn
105	0.011	0.500	0.247	0.079	0.064	0.566	#21	0.076	Not Required	Pass
106	0.011	0.812	0.057	0.080	0.003	0.865	#21	0.046	Not Required	Pass
107	0.011	0.504	0.232	0.080	0.061	0.570	#21	0.076	Not Required	Pass
108	0.004	0.080	0.282	0.050	0.024	0.362	#21	0.102	Not Required	Pass
109	0.028	0.074	0.051	0.001	0.000	0.135	#21	0.206	Not Required	Pass
110	0.011	0.801	0.220	0.080	0.047	0.942	#21	0.082	Not Required	Pass

111	0.006	0.061	0.291	0.050	0.024	0.354	#21	0.102	Not Required	Pass
112	0.006	0.569	0.117	0.123	0.019	0.682	#21	0.036	Not Required	Pass
113	0.008	0.284	0.658	0.064	0.030	0.906	#21	0.306	Not Required	Pass
114	0.012	0.322	0.649	0.065	0.030	0.935	#21	0.306	Not Required	Pass
115	0.016	0.618	0.334	0.052	0.024	0.960	#21	0.644	Not Required	Warn
116	0.004	0.590	0.330	0.054	0.024	0.920	#21	0.644	Not Required	Pass
201	0.270	0.861	0.009	0.036	0.001	0.951	#13	0.672	Not Required	Warn
202	0.006	0.569	0.117	0.123	0.019	0.682	#21	0.036	Not Required	Pass
203	0.011	0.812	0.057	0.080	0.003	0.865	#21	0.046	Not Required	Pass
204	0.011	0.801	0.220	0.080	0.047	0.942	#21	0.082	Not Required	Pass
205	0.011	0.504	0.232	0.080	0.061	0.570	#21	0.076	Not Required	Pass
206	0.011	0.806	0.055	0.080	0.001	0.867	#21	0.046	Not Required	Pass
207	0.011	0.500	0.247	0.079	0.064	0.566	#21	0.076	Not Required	Pass
208	0.003	0.068	0.319	0.054	0.024	0.388	#21	0.102	Not Required	Pass
209	0.028	0.074	0.051	0.001	0.000	0.135	#21	0.206	Not Required	Pass
210	0.011	0.815	0.235	0.081	0.049	0.964	#21	0.082	Not Required	Warn
211	0.006	0.067	0.328	0.052	0.024	0.374	#21	0.102	Not Required	Pass
212	0.006	0.572	0.116	0.124	0.017	0.685	#21	0.036	Not Required	Pass
213	0.008	0.284	0.658	0.064	0.030	0.906	#21	0.306	Not Required	Pass
214	0.012	0.322	0.648	0.065	0.030	0.935	#21	0.306	Not Required	Pass
215	0.016	0.492	0.333	0.050	0.024	0.830	#21	0.644	Not Required	Pass
216	0.006	0.421	0.327	0.050	0.024	0.749	#21	0.644	Not Required	Pass
301	0.217	0.774	0.086	0.029	0.006	0.809	#13	0.672	Not Required	Pass
302	0.003	0.520	0.098	0.110	0.017	0.609	#21	0.054	Not Required	Pass
303	0.011	0.718	0.088	0.072	0.017	0.811	#21	0.046	Not Required	Pass
304	0.011	0.682	0.214	0.068	0.045	0.798	#21	0.082	Not Required	Pass
305	0.011	0.446	0.231	0.071	0.059	0.502	#21	0.076	Not Required	Pass
306	0.007	0.584	0.042	0.057	0.010	0.600	#21	0.046	Not Required	Pass
307	0.007	0.362	0.120	0.058	0.032	0.394	#21	0.076	Not Required	Pass
308	0.000	0.056	0.154	0.025	0.012	0.210	#21	Not Required	Not Required	Pass
309	0.019	0.064	0.077	0.002	0.004	0.150	#21	0.206	Not Required	Pass
310	0.006	0.579	0.130	0.058	0.029	0.678	#21	0.082	Not Required	Pass
311	0.000	0.056	0.154	0.025	0.012	0.210	#21	Not Required	Not Required	Pass
312	0.004	0.386	0.085	0.087	0.015	0.464	#21	0.036	Not Required	Pass
313	0.008	0.199	0.619	0.059	0.029	0.731	#21	0.204	Not Required	Pass
314	0.009	0.187	0.606	0.057	0.029	0.718	#24	0.306	Not Required	Pass
315	0.016	0.633	0.333	0.048	0.024	0.969	#21	0.644	Not Required	Warn
316	0.004	0.620	0.324	0.045	0.024	0.945	#21	0.644	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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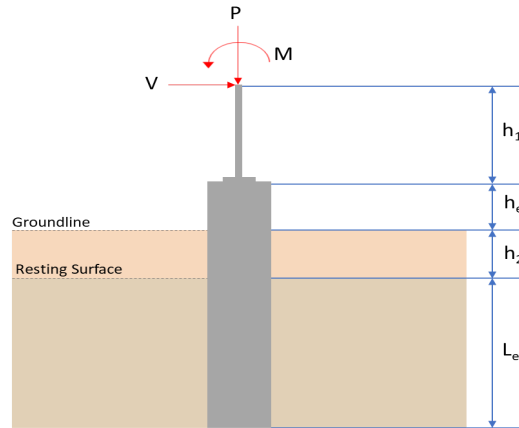
## SkyCiv Foundation Design

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

$L = 5.75$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	9.165	15.132
$V_x$ (kip)	-1.312	-2.206
$V_z$ (kip)	0.272	0.452
$M_x$ (kipft)	1.051	1.762
$M_z$ (kipft)	18.281	32.724

### 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,

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

$$H_o = \frac{(-1.312 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.20892 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.281 \text{ kipft}) + ((-1.312 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.911 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.4763 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.272 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.043312 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.051 \text{ kipft}) + ((0.272 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16736 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7369 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.4763 \text{ ft}), (2.7369 \text{ ft})]$$

$$L_{e,req} = 5.476 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.476 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95235$$

Status: **PASS**  
Ratio: **0.950**

### 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

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

$$q = \frac{(9.165 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.57281 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.57281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28641$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.20892 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.911 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.911 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.911 \text{ kipft/ft})) + (4 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9367 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.911 \text{ kipft/ft})) + (3 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.911 \text{ kipft/ft})) + (2 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.23164 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.911 \text{ kipft/ft})) + ((-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.83854 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9367 \text{ ft})}{2}$$

$$p_a = 0.29525 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23164 \text{ kip/ft}^2)}{(0.29525 \text{ kip/ft}^2)}$$

$$Ratio = 0.78456$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.83854 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.97222$$

Status: **PASS**  
Ratio: **0.780**

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

#### Considering z-direction:

$H_o = 0.043312 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.16736 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.16736 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.043312 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.16736 \text{ kipft/ft})) + (4 \times (0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.072 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.16736 \text{ kipft/ft})) + (3 \times (0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.16736 \text{ kipft/ft})) + (2 \times (0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.045512 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.16736 \text{ kipft/ft})) + ((0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.10594 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.072 \text{ ft})}{2}$$

$$p_a = 0.3054 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.045512 \text{ kip/ft}^2)}{(0.3054 \text{ kip/ft}^2)}$$

$$Ratio = 0.14903$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

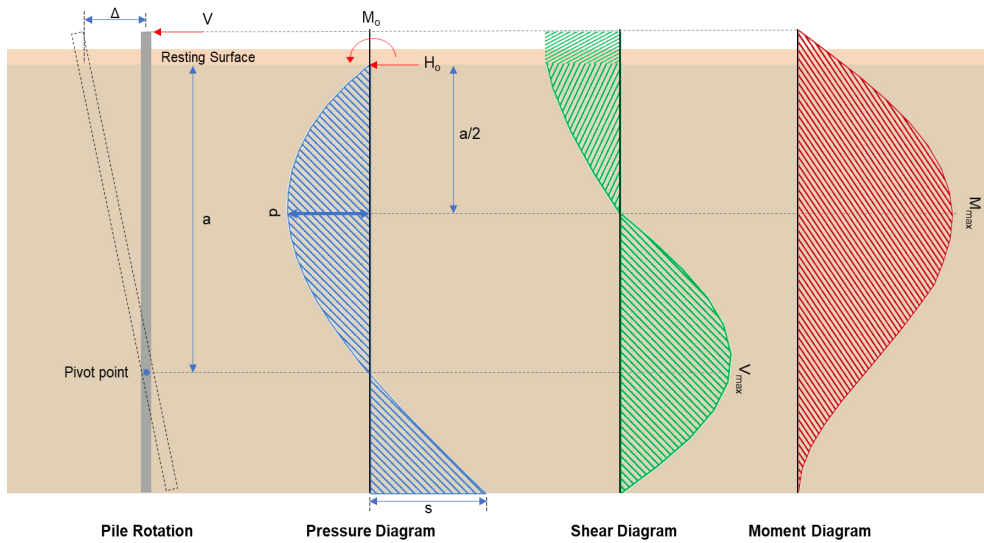
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.10594 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.12283$$

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

Status: **PASS**  
Ratio: **0.120**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.206 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.35127 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(32.724 \text{ kipft}) + ((-2.206 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.2108 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.2108 \text{ kipft/ft})}{(-0.35127 \text{ kip/ft})}$$

$$E = 14.834 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.2108 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.2108 \text{ kipft/ft})) + (4 \times (-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (5.2108 \text{ kipft/ft})) + (4 \times (-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9317 \text{ ft}$$

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

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

$$V_{max} = ((-0.35127 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (14.834 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9317 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (14.834 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9317 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.3452 \text{ kip}$$

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

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

$$M_{max} = ((-0.35127 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(14.834 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (14.834 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (14.834 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.38 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.452 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.071975 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.762 \text{ kipft}) + ((0.452 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28057 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.28057 \text{ kipft/ft})}{(0.071975 \text{ kip/ft})}$$

$$E = 3.8982 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.28057 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.071975 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.28057 \text{ kipft/ft})) + (4 \times (0.071975 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0709 \text{ ft}$$

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

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

$$V_{max} = ((0.071975 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8982 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.0709 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.8982 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.0709 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.53635 \text{ kip}$$

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

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

$$M_{max} = ((0.071975 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(3.8982 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0709 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.8982 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.0709 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8982 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.0709 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.3937 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(15.132 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.093 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.093 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(15.132 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0056564$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 15.132 \text{ kip} \rightarrow 15132 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15132 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.5 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.5 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.5 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.5 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.41 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.3452 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.3452 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.06593$$

**Considering z-direction:**

$V_{max} = 0.53635 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.53635 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.0048143$$

Status: **PASS**  
 Ratio: **0.070**

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:**

$M_{max} = 20.38 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(20.38 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.081651$$

Status: **PASS**  
 Ratio: **0.080**

**Considering z-direction:**

$M_{max} = 1.3937 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(1.3937 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0055838$$

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

REFERENCES	CALCULATIONS	RESULTS
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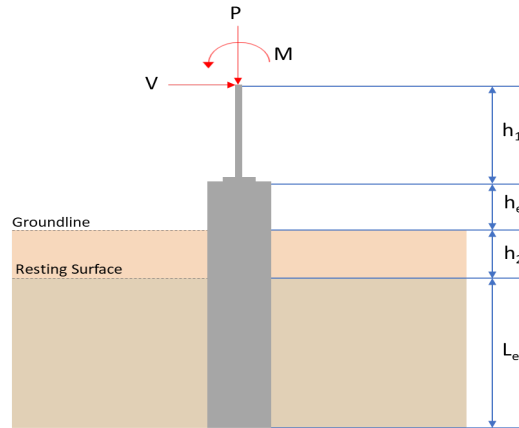
## SkyCiv Foundation Design

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

$L = 5.75$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	9.165	15.132
$V_x$ (kip)	-1.312	-2.206
$V_z$ (kip)	-0.272	-0.452
$M_x$ (kipft)	-1.051	-1.763
$M_z$ (kipft)	18.281	32.725

### 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,

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

$$H_o = \frac{(-1.312 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.20892 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.281 \text{ kipft}) + ((-1.312 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.911 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.4763 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.272 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.043312 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.051 \text{ kipft}) + ((-0.272 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16736 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0131 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(5.4763 \text{ ft}), (2.0131 \text{ ft})]$$

$$L_{e,req} = 5.476 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$Ratio = \frac{(5.476 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.95235$$

Status: **PASS**  
Ratio: **0.950**

### 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

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

$$q = \frac{(9.165 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.57281 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.57281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28641$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 1.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.20892 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.911 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.911 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.911 \text{ kipft/ft})) + (4 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9367 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.911 \text{ kipft/ft})) + (3 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.911 \text{ kipft/ft})) + (2 \times (-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.23164 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.911 \text{ kipft/ft})) + ((-0.20892 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.83854 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9367 \text{ ft})}{2}$$

$$p_a = 0.29525 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23164 \text{ kip/ft}^2)}{(0.29525 \text{ kip/ft}^2)}$$

$$Ratio = 0.78456$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.83854 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.97222$$

Status: **PASS**  
Ratio: **0.780**

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

#### Considering z-direction:

$H_o = -0.043312 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.16736 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.16736 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.043312 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.16736 \text{ kipft/ft})) + (4 \times (-0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.072 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.16736 \text{ kipft/ft})) + (3 \times (-0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.16736 \text{ kipft/ft})) + (2 \times (-0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.034409 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.16736 \text{ kipft/ft})) + ((-0.043312 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.015547 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.072 \text{ ft})}{2}$$

$$p_a = 0.3054 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.034409 \text{ kip/ft}^2)}{(0.3054 \text{ kip/ft}^2)}$$

$$Ratio = 0.11267$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

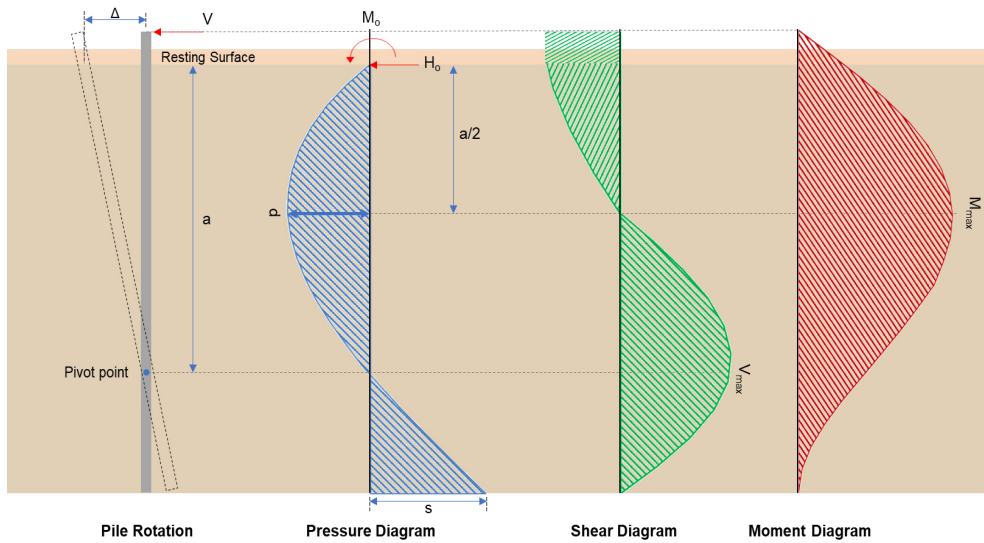
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.015547 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.018025$$

Status: **PASS**  
Ratio: **0.110**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.206 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.35127 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(32.725 \text{ kipft}) + ((-2.206 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.211 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.211 \text{ kipft/ft})}{(-0.35127 \text{ kip/ft})}$$

$$E = 14.835 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.211 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times 5.211) + (4 \times (-0.35127) \times 5.75)}$$

$$a = \frac{(-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (5.211 \text{ kip/ft})) + (4 \times (-0.35127 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9317 \text{ ft}$$

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

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

$$V_{max} = ((-0.35127 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (14.835 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9317 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (14.835 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9317 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.3454 \text{ kip}$$

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

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

$$M_{max} = ((-0.35127 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(14.835 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (14.835 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (14.835 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9317 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 20.381 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.452 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.071975 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.763 \text{ kipft}) + ((-0.452 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28073 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.28073 \text{ kipft/ft})}{(-0.071975 \text{ kip/ft})}$$

$$E = 3.9004 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.28073 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.071975 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.28073 \text{ kipft/ft})) + (4 \times (-0.071975 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0708 \text{ ft}$$

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

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

$$V_{max} = ((-0.071975 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.9004 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.0708 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.9004 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.0708 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.53655 \text{ kip}$$

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

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

$$M_{max} = ((-0.071975 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(3.9004 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0708 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.9004 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.0708 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.9004 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.0708 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.3943 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\left( \frac{15.132 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.093 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.093 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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_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{(15.132 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0056564</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.010</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 15.132 \text{ kip} \rightarrow 15132 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(15132 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.5 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.5 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.5 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.5 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.41 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.3454 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.3454 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.065932$$

Status: **PASS**  
Ratio: **0.070**

**Considering z-direction:**

$V_{max} = 0.53655 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.53655 \text{ kip})}{(111.41 \text{ kip})}$$

$$Ratio = 0.0048161$$

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:**

$M_{max} = 20.381 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(20.381 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.081654$$

Status: **PASS**  
Ratio: **0.080**

**Considering z-direction:**

$M_{max} = 1.3943 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.3943 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0055861$$

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

REFERENCES	CALCULATIONS	RESULTS
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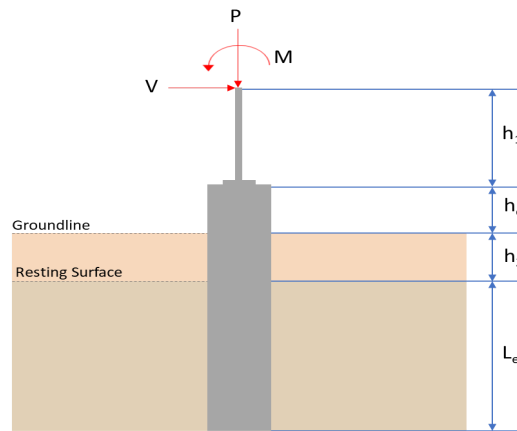
## SkyCiv Foundation Design

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

$L = 6$  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)	11.390	18.853
$V_x$ (kip)	-1.600	-2.677
$V_z$ (kip)	-0.028	-0.049
$M_x$ (kipft)	-0.106	-0.188
$M_z$ (kipft)	20.582	36.400

### 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,

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

$$H_o = \frac{(-1.6 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(20.582 \text{ kipft}) + ((-1.6 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2774 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6088 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.028 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0044586 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.106 \text{ kipft}) + ((-0.028 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016879 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0248 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.6088 \text{ ft}), (1.0248 \text{ ft})]$$

$$L_{e,req} = 5.609 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.609 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.93483$$

Status: **PASS**  
Ratio: **0.930**

**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

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

$$q = \frac{(11.39 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71188 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71188 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35594$$

Status: **PASS**  
Ratio: **0.360**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.25478 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.2774 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.2774 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.2774 \text{ kipft/ft})) + (4 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1186 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2774 \text{ kipft/ft})) + (3 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.2774 \text{ kipft/ft})) + (2 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.22341 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.2774 \text{ kipft/ft})) + ((-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.83769 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1186 \text{ ft})}{2}$$

$$p_a = 0.30889 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22341 \text{ kip/ft}^2)}{(0.30889 \text{ kip/ft}^2)}$$

$$Ratio = 0.72326$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.83769 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.93076$$

Status: **PASS**  
Ratio: **0.720**

Status: **PASS**  
Ratio: **0.930**

#### Considering z-direction:

$H_o = -0.0044586 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.016879 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.016879 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.0044586 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.016879 \text{ kipft/ft})) + (4 \times (-0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2569 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.016879 \text{ kipft/ft})) + (3 \times (-0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 [(3 \times (0.016879 \text{ kipft/ft})) + (2 \times (-0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.0011795 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 [(2 \times (0.016879 \text{ kipft/ft})) + ((-0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.0011677 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2569 \text{ ft})}{2}$$

$$p_a = 0.31927 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0011795 \text{ kip/ft}^2)}{(0.31927 \text{ kip/ft}^2)}$$

$$Ratio = -0.0036945$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

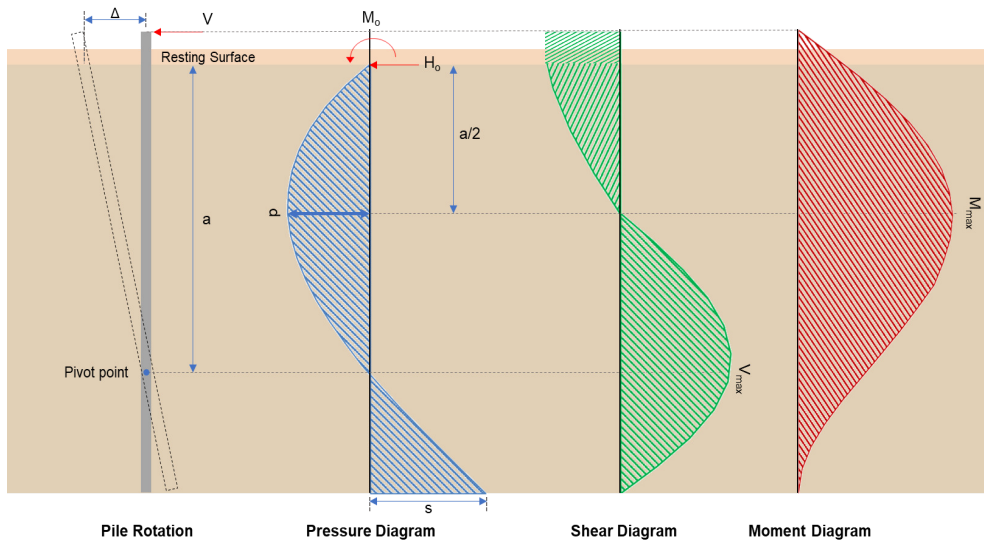
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0011677 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.0012975$$

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,

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

$$H_o = \frac{(-2.677 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42627 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.4 \text{ kipft}) + ((-2.677 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.7962 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.7962 \text{ kipft/ft})}{(-0.42627 \text{ kip/ft})}$$

$$E = 13.597 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.7962 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.7962 \text{ kipft/ft})) + (4 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.42627 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.7962 \text{ kipft/ft})) + (4 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1137 \text{ ft}$$

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

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

$$V_{max} = ((-0.42627 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (13.597 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1137 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (13.597 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1137 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.9649 \text{ kip}$$

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

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

$$M_{max} = ((-0.42627 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(13.597 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (13.597 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (13.597 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.963 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.049 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0078025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.188 \text{ kipft}) + ((-0.049 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.029936 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.029936 \text{ kipft/ft})}{(-0.0078025 \text{ kip/ft})}$$

$$E = 3.8367 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.029936 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.0078025 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.029936 \text{ kipft/ft})) + (4 \times (-0.0078025 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2552 \text{ ft}$$

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

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

$$V_{max} = ((-0.0078025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.2552 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.2552 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.056035 \text{ kip}$$

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

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

$$M_{max} = ((-0.0078025 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(3.8367 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.15134 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.853 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.97 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.97 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(18.853 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0070474</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.010</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.853 \text{ kip} \rightarrow 18853 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18853 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (121 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((121 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.73 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.9649 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(7.9649 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.071286$$

Status: **PASS**  
Ratio: **0.070**

**Considering z-direction:**

$V_{max} = 0.056035 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.056035 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.00050152$$

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:**

$M_{max} = 22.963 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(22.963 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.091997$$

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

**Considering z-direction:**

$M_{max} = 0.15134 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.15134 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00060635$$

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

REFERENCES	CALCULATIONS	RESULTS
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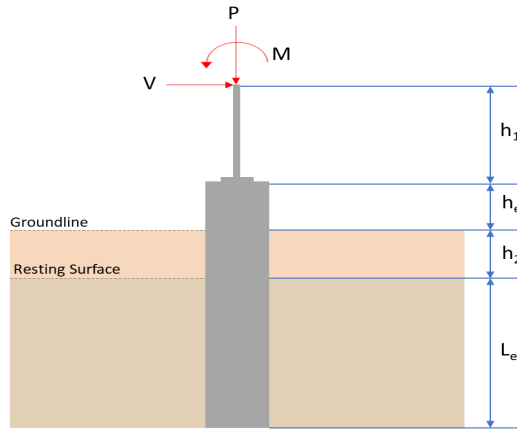
## SkyCiv Foundation Design

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

$L = 6$  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)	11.390	18.853
$V_x$ (kip)	-1.600	-2.677
$V_z$ (kip)	0.028	0.049
$M_x$ (kipft)	0.106	0.188
$M_z$ (kipft)	20.582	36.401

### 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,

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

$$H_o = \frac{(-1.6 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25478 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(20.582 \text{ kipft}) + ((-1.6 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2774 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6088 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.028 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0044586 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.106 \text{ kipft}) + ((0.028 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016879 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1859 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.6088 \text{ ft}), (1.1859 \text{ ft})]$$

$$L_{e,req} = 5.609 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.609 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.93483$$

Status: **PASS**  
Ratio: **0.930**

### 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

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

$$q = \frac{(11.39 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.71188 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.71188 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35594$$

Status: **PASS**  
Ratio: **0.360**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.25478 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.2774 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.2774 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.2774 \text{ kipft/ft})) + (4 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1186 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2774 \text{ kipft/ft})) + (3 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.2774 \text{ kipft/ft})) + (2 \times (-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.22341 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.2774 \text{ kipft/ft})) + ((-0.25478 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.83769 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1186 \text{ ft})}{2}$$

$$p_a = 0.30889 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22341 \text{ kip/ft}^2)}{(0.30889 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.72326$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.83769 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93076$$

Status: **PASS**  
Ratio: **0.720**

Status: **PASS**  
Ratio: **0.930**

#### Considering z-direction:

$H_o = 0.0044586 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.016879 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.016879 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.0044586 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.016879 \text{ kipft/ft})) + (4 \times (0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2569 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.016879 \text{ kipft/ft})) + (3 \times (0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.016879 \text{ kipft/ft})) + (2 \times (0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.0043683 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.016879 \text{ kipft/ft})) + ((0.0044586 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.010085 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2569 \text{ ft})}{2}$$

$$p_a = 0.31927 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0043683 \text{ kip/ft}^2)}{(0.31927 \text{ kip/ft}^2)}$$

$$Ratio = 0.013682$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

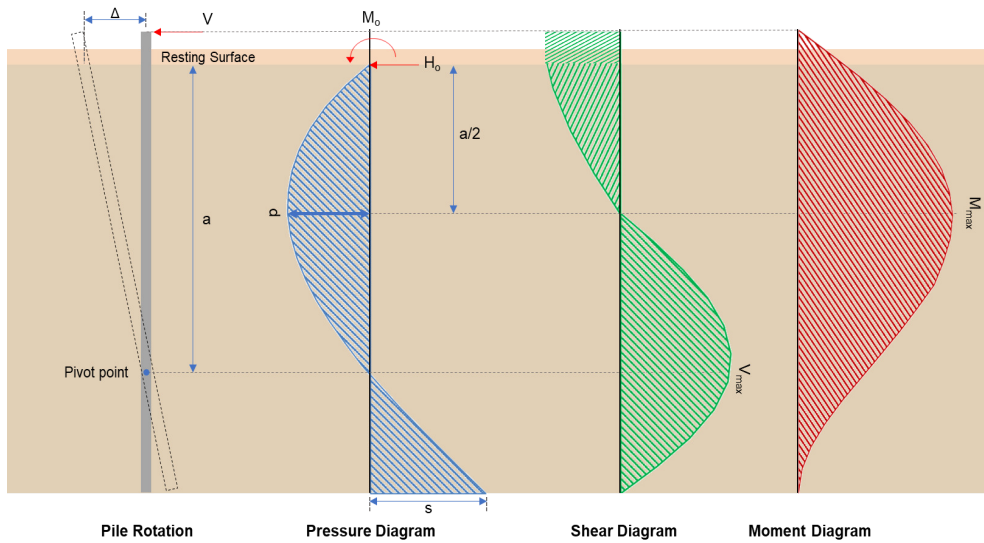
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.010085 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.011205$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.677 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42627 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.401 \text{ kipft}) + ((-2.677 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.7963 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.7963 \text{ kipft/ft})}{(-0.42627 \text{ kip/ft})}$$

$$E = 13.598 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.7963 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.7963 \text{ kipft/ft})) + (4 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.42627 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.7963 \text{ kip/ft})) + (4 \times (-0.42627 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1137 \text{ ft}$$

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

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

$$V_{max} = ((-0.42627 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (13.598 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1137 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (13.598 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1137 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.965 \text{ kip}$$

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

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

$$M_{max} = ((-0.42627 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(13.598 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (13.598 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (13.598 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1137 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.963 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.049 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0078025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.188 \text{ kipft}) + ((0.049 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.029936 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.029936 \text{ kipft/ft})}{(0.0078025 \text{ kip/ft})}$$

$$E = 3.8367 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.029936 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.0078025 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.029936 \text{ kipft/ft})) + (4 \times (0.0078025 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2552 \text{ ft}$$

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

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

$$V_{max} = ((0.0078025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.2552 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.2552 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.056035 \text{ kip}$$

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

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

$$M_{max} = ((0.0078025 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(3.8367 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8367 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.2552 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.15134 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.853 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.97 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.97 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(18.853 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0070474</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.010</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.853 \text{ kip} \rightarrow 18853 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18853 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (121 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((121 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.73 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.965 \text{ kip}$  - Maximum shear force in the x-direction,  
 $Ratio$  - Capacity

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

$$Ratio = \frac{(7.965 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.071288$$

Status: **PASS**  
Ratio: **0.070**

**Considering z-direction:**

$V_{max} = 0.056035 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.056035 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.00050152$$

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:**

$M_{max} = 22.963 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(22.963 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.092$$

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

**Considering z-direction:**

$M_{max} = 0.15134 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.15134 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00060635$$

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