

# Project Details



**Project Name:** MTSOLAR\_G69IFFKL0HIH

**Date:** Tue Nov 19 2024

**Location:** 2008 Olympia St, Idaho Falls, ID 83402, USA

**Number of Modules:** 45

**Unique ID:** 4P-19.75-8TOP-XD-24-L-5Hx9W-K207

**Number of Poles:** 4

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

**Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	18.19 ft
<b>Array Dimensions E/W</b>	72.75 ft
<b>Winter Tilt Angle</b>	35
<b>Front Edge Clearance</b>	5 ft

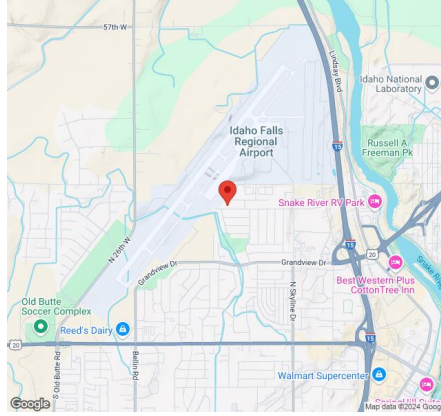
## MT Solar Bill of Materials (4P-19.75-8TOP-XD-24-L-5Hx9W-K207)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-24	24IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	9

## Rail Bill of Materials

Part	Qty
Rails (216in)	18
Rail Attachment	72
Module Mid Clamp	72
Module End Clamp	36
Ground Lug	9

## Site Details:



**Site Address:** 2008 Olympia St, Idaho Falls, ID 83402, USA

### Array Specification

<b>Duty Classification:</b>	XD
<b>Module Width:</b>	43.15 in
<b>Module Length:</b>	96.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	9
<b>Total Number of Modules:</b>	45
<b>Winter Tilt Angle:</b>	35
<b>Front Edge Clearance:</b>	5
<b>Total Array Height at Tilt:</b>	15.43 ft
<b>Total Frame Length:</b>	70.75 ft
<b>Frame Weight:</b>	4877 lbs
<b>Array Dimensions N/S:</b>	18.19 ft
<b>Array Dimensions E/W:</b>	72.75 ft
<b>Rail Length:</b>	218.25 in
<b>Rail Spacing:</b>	4.04 ft

### Support Specifications

<b>Pole Size:</b>	8in Pipe Sch 40
<b>Pole Length above Grade:</b>	10.22 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	19.75 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.75 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 5.75 ft
<b>Foundation Volume:</b>	14.222 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	2008 Olympia St, Idaho Falls, ID 83402, USA
<b>Wind Speed:</b>	99 mph

**Snow Load:**

64.33 psf

### **Design Disclaimer**

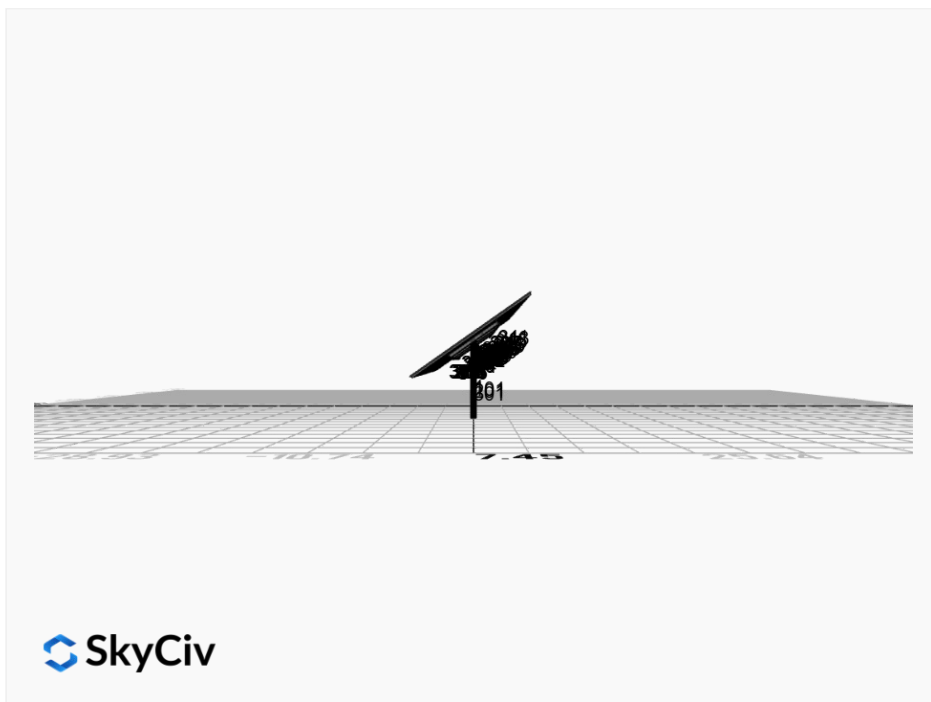
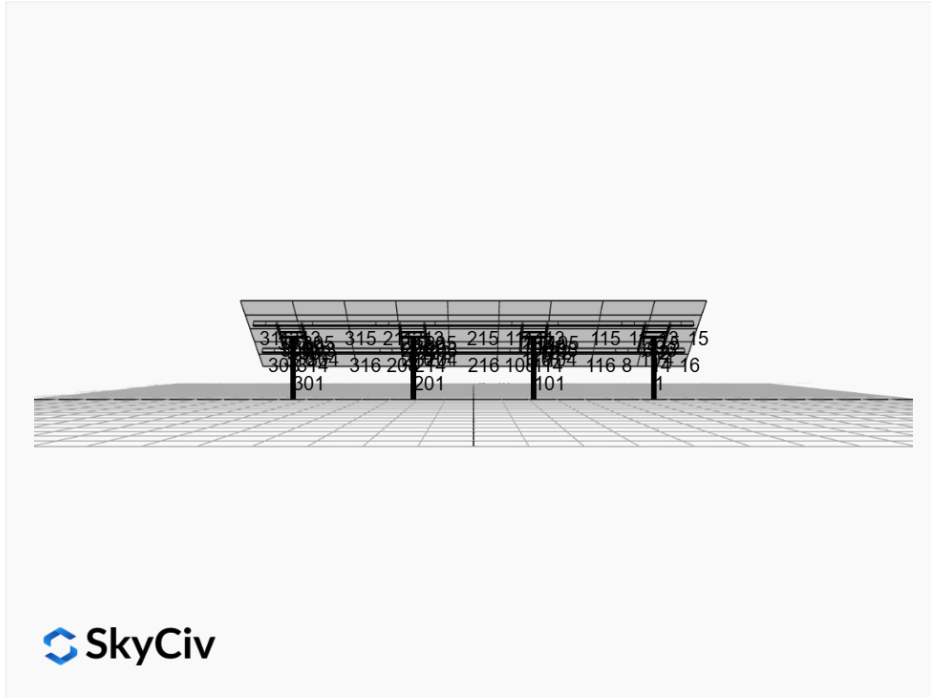
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

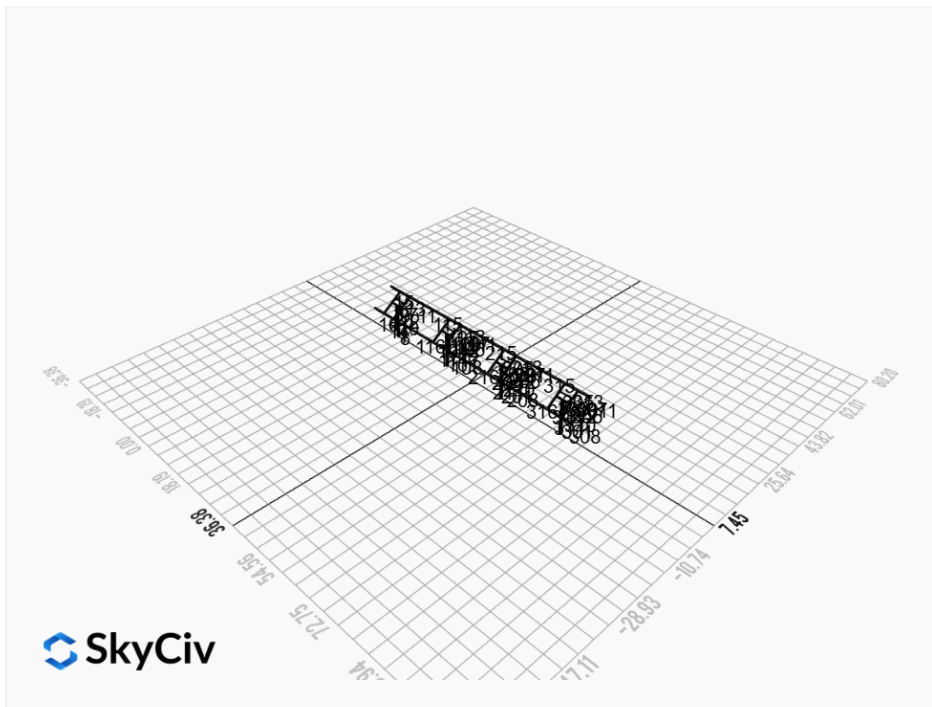
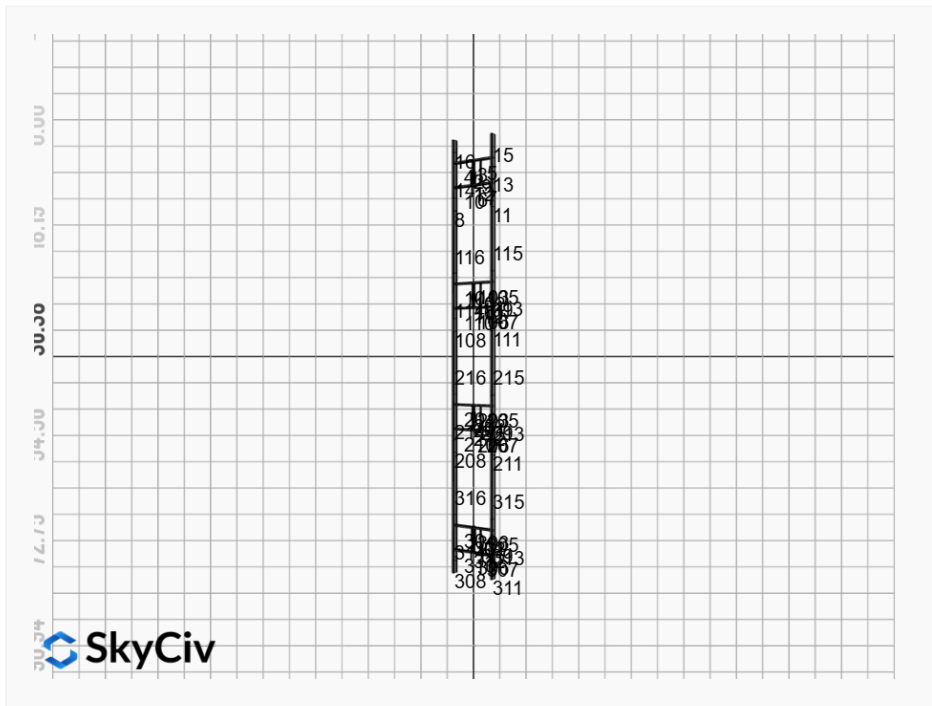
## AutoDesigner Input

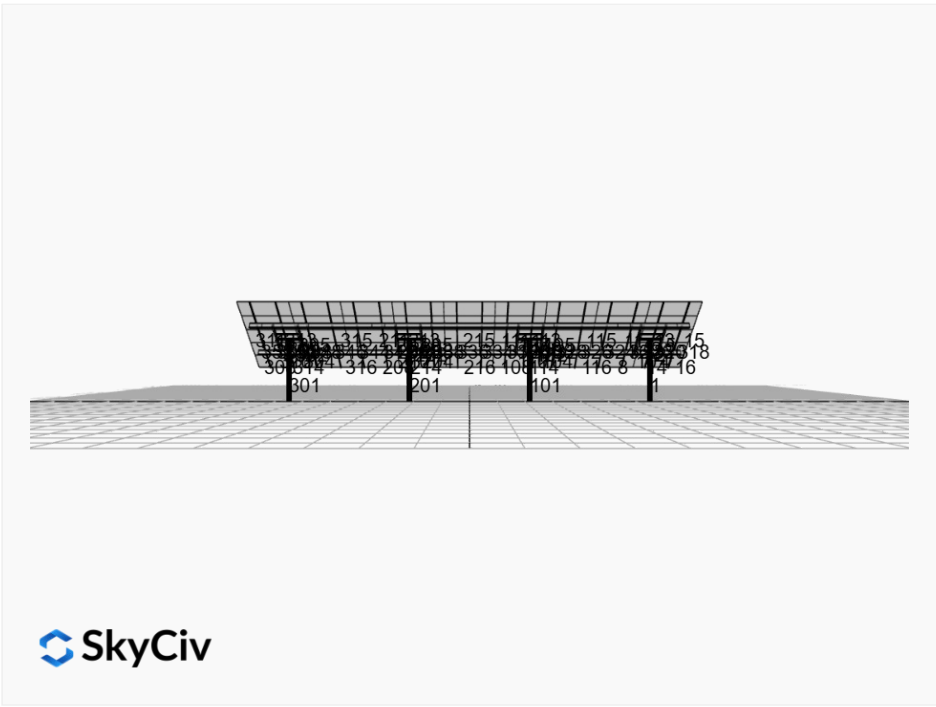
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## Design Notes:

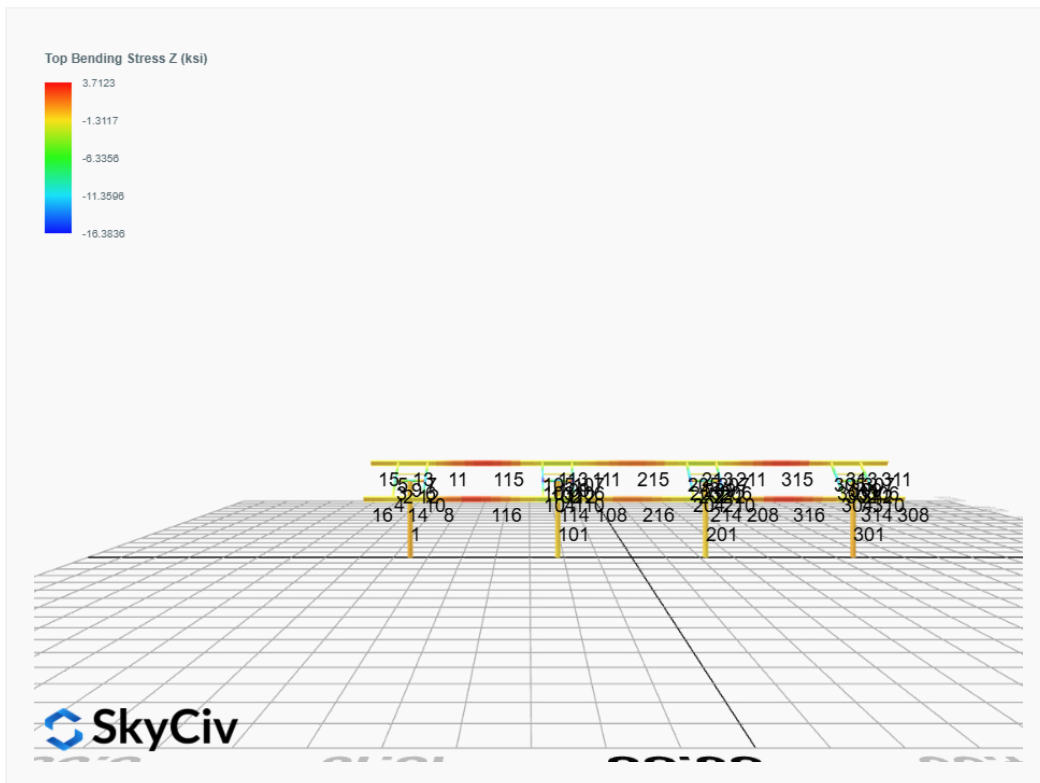
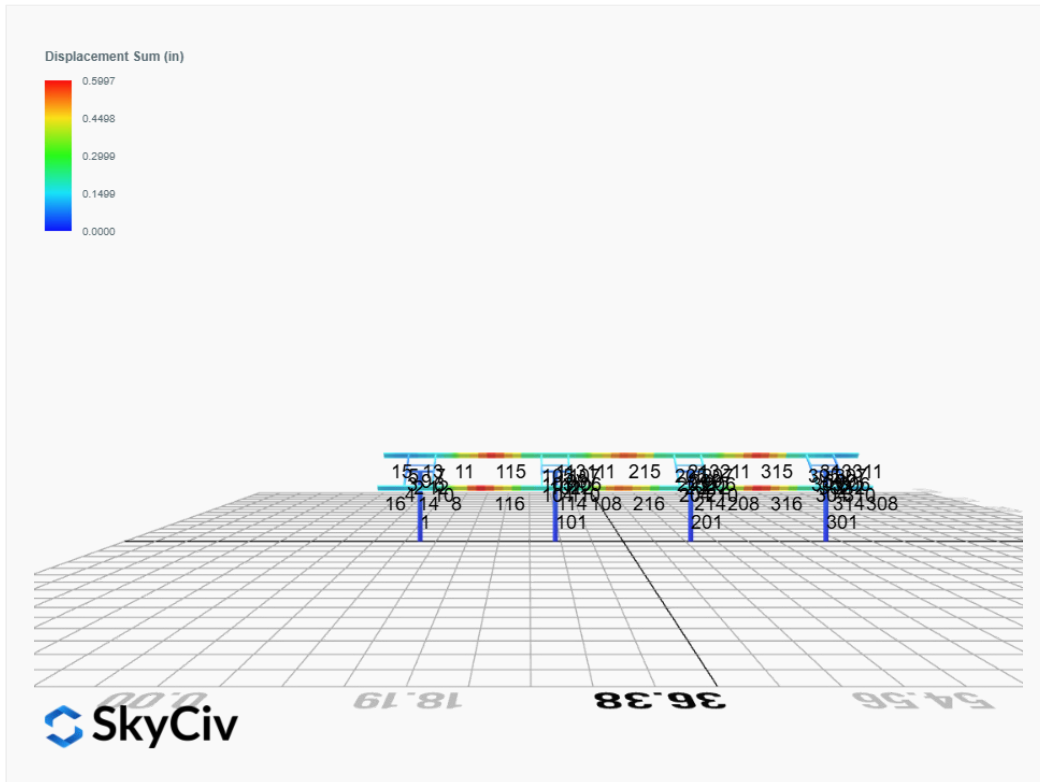
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



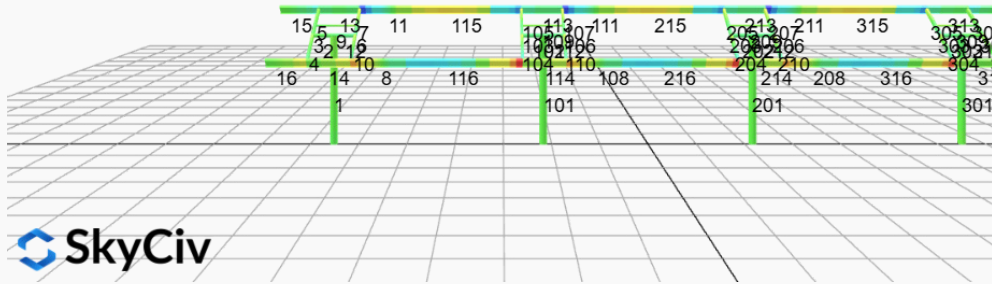
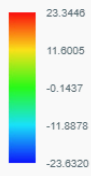




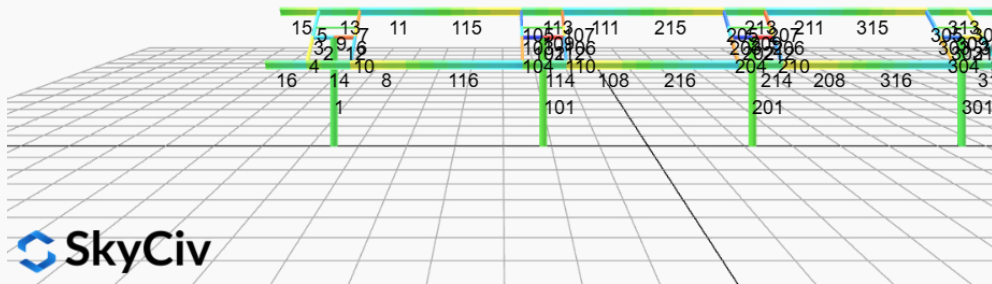
# FEM Results (Envelope Worst Case for each member)

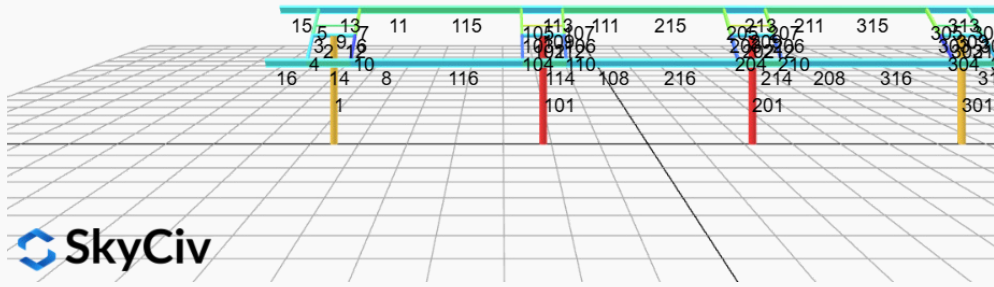
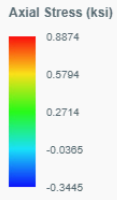


Top Bending Stress Y (ksi)



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	0.0306	2.1787	0.0797	0.2343	-0.0477	-0.2635
ULS: 2. D + L	0.0306	2.1787	0.0797	0.2343	-0.0477	-0.2635
ULS: 3. D + (S or Lr or R)	0.1364	7.7742	0.3576	1.0541	-0.2152	-1.2496
ULS: 3. D + (S or Lr or R)	0.0306	2.1787	0.0797	0.2343	-0.0477	-0.2635
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1099	6.3753	0.2881	0.8491	-0.1733	-1.0031
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0306	2.1787	0.0797	0.2343	-0.0477	-0.2635
ULS: 5b. D + 0.7E	0.0306	2.1787	0.0797	0.2343	-0.0477	-0.2635
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1099	6.3753	0.2881	0.8491	-0.1733	-1.0031
ULS: 8. 0.6D + 0.7E	0.0183	1.3072	0.0478	0.1406	-0.0286	-0.1581
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8129	4.7690	0.2650	0.7504	-0.5462	19.1631
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8129	4.7690	0.2650	0.7504	-0.5462	19.1631
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5838	-0.0052	-0.0736	-0.1921	0.3648	-16.1537
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.3436	0.3473	-0.0673	-0.1740	0.3539	-18.9863
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2727	8.3180	0.4271	1.2362	-0.5472	13.5669
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2727	8.3180	0.4271	1.2362	-0.5472	13.5669
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2748	4.7374	0.1732	0.5293	0.1361	-12.9207
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0947	5.0017	0.1779	0.5429	0.1279	-15.0451
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	4.1214	0.2187	0.6214	-0.4216	14.3064
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	4.1214	0.2187	0.6214	-0.4216	14.3064
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1955	0.5408	-0.0353	-0.0855	0.2617	-12.1812
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0153	0.8051	-0.0305	-0.0720	0.2535	-14.3056
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8251	3.8975	0.2331	0.6567	-0.5271	19.2685
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8251	3.8975	0.2331	0.6567	-0.5271	19.2685
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5715	-0.8766	-0.1055	-0.2859	0.3839	-16.0483
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3313	-0.5242	-0.0991	-0.2678	0.3730	-18.8809

### 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	13.7276
Shear X	-3.0724
Shear Z	0.7013
Moment X	2.0488
Moment Y (Twist)	0.9797
Moment Z	32.5238

### 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	8.3180
Shear X	-1.8251
Shear Z	0.4271
Moment X	1.2362
Moment Y (Twist)	0.5472
Moment Z	19.2685

## 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	-0.0306	2.7151	-0.0048	-0.0145	0.0111	0.3214
ULS: 2. D + L	-0.0306	2.7151	-0.0048	-0.0145	0.0111	0.3214
ULS: 3. D + (S or Lr or R)	-0.1364	10.1682	-0.0211	-0.0642	0.0492	1.3984
ULS: 3. D + (S or Lr or R)	-0.0306	2.7151	-0.0048	-0.0145	0.0111	0.3214
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1099	8.3049	-0.0170	-0.0518	0.0397	1.1292

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0306	2.7151	-0.0048	-0.0145	0.0111	0.3214
ULS: 5b. D + 0.7E	-0.0306	2.7151	-0.0048	-0.0145	0.0111	0.3214
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1099	8.3049	-0.0170	-0.0518	0.0397	1.1292
ULS: 8. 0.6D + 0.7E	-0.0183	1.6290	-0.0029	-0.0087	0.0067	0.1928
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4476	6.2094	0.0056	0.0117	-0.0397	25.4069
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.4476	6.2094	0.0056	0.0117	-0.0397	25.4069
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0110	-0.2350	-0.0119	-0.0321	0.0489	-20.1362
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.6521	0.2683	-0.0214	-0.0587	0.0780	-23.3284
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9227	10.9257	-0.0092	-0.0322	0.0016	19.9433
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9227	10.9257	-0.0092	-0.0322	0.0016	19.9433
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4213	6.0924	-0.0224	-0.0650	0.0680	-14.2140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1521	6.4698	-0.0295	-0.0850	0.0898	-16.6082
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8433	5.3358	0.0030	0.0051	-0.0270	19.1355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8433	5.3358	0.0030	0.0051	-0.0270	19.1355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5006	0.5025	-0.0101	-0.0277	0.0395	-15.0218
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2314	0.8800	-0.0173	-0.0476	0.0612	-17.4159
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.4354	5.1234	0.0075	0.0175	-0.0441	25.2783
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.4354	5.1234	0.0075	0.0175	-0.0441	25.2783
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0233	-1.3210	-0.0100	-0.0263	0.0445	-20.2648
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.6643	-0.8178	-0.0195	-0.0529	0.0735	-23.4569

### 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	18.0933
Shear X	-4.1156
Shear Z	-0.0477
Moment X	-0.1390
Moment Y (Twist)	0.1496
Moment Z	43.5033

### 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	10.9257
Shear X	-2.4476
Shear Z	-0.0295
Moment X	-0.0850
Moment Y (Twist)	0.0898
Moment Z	25.4069

### 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	-0.0306	2.7151	0.0048	0.0145	-0.0111	0.3214
ULS: 2. D + L	-0.0306	2.7151	0.0048	0.0145	-0.0111	0.3214
ULS: 3. D + (S or Lr or R)	-0.1364	10.1682	0.0211	0.0641	-0.0488	1.3984
ULS: 3. D + (S or Lr or R)	-0.0306	2.7151	0.0048	0.0145	-0.0111	0.3214
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1099	8.3049	0.0170	0.0517	-0.0394	1.1292
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0306	2.7151	0.0048	0.0145	-0.0111	0.3214
ULS: 5b. D + 0.7E	-0.0306	2.7151	0.0048	0.0145	-0.0111	0.3214
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1099	8.3049	0.0170	0.0517	-0.0394	1.1292
ULS: 8. 0.6D + 0.7E	-0.0183	1.6290	0.0029	0.0087	-0.0067	0.1928
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4476	6.2094	-0.0056	-0.0117	0.0397	25.4069
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.4476	6.2094	-0.0056	-0.0117	0.0397	25.4069
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0110	-0.2350	0.0119	0.0321	-0.0489	-20.1362
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.6521	0.2683	0.0214	0.0587	-0.0779	-23.3284

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	-1.9227	10.9257	0.0093	0.0321	-0.0013	19.9432
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9227	10.9257	0.0093	0.0321	-0.0013	19.9432
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4213	6.0924	0.0224	0.0648	-0.0677	-14.2140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1521	6.4698	0.0295	0.0848	-0.0895	-16.6082
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8433	5.3358	-0.0030	-0.0051	0.0270	19.1355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8433	5.3358	-0.0030	-0.0051	0.0270	19.1355
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5006	0.5025	0.0101	0.0277	-0.0395	-15.0218
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2314	0.8800	0.0173	0.0476	-0.0612	-17.4159
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.4354	5.1234	-0.0075	-0.0175	0.0441	25.2783
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.4354	5.1234	-0.0075	-0.0175	0.0441	25.2783
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0233	-1.3210	0.0100	0.0263	-0.0445	-20.2648
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.6643	-0.8178	0.0195	0.0529	-0.0735	-23.4569

### 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	18.0933
Shear X	-4.1156
Shear Z	0.0477
Moment X	0.1390
Moment Y (Twist)	0.1493
Moment Z	43.5034

### 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	10.9257
Shear X	-2.4476
Shear Z	0.0295
Moment X	0.0848
Moment Y (Twist)	0.0895
Moment Z	25.4069

### 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	0.0306	2.1787	-0.0797	-0.2343	0.0477	-0.2635
ULS: 2. D + L	0.0306	2.1787	-0.0797	-0.2343	0.0477	-0.2635
ULS: 3. D + (S or Lr or R)	0.1364	7.7742	-0.3576	-1.0546	0.2155	-1.2493
ULS: 3. D + (S or Lr or R)	0.0306	2.1787	-0.0797	-0.2343	0.0477	-0.2635
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.1099	6.3753	-0.2882	-0.8495	0.1736	-1.0028
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0306	2.1787	-0.0797	-0.2343	0.0477	-0.2635
ULS: 5b. D + 0.7E	0.0306	2.1787	-0.0797	-0.2343	0.0477	-0.2635
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.1099	6.3753	-0.2882	-0.8495	0.1736	-1.0028
ULS: 8. 0.6D + 0.7E	0.0183	1.3072	-0.0478	-0.1406	0.0286	-0.1581
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8129	4.7690	-0.2650	-0.7504	0.5462	19.1631
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8129	4.7690	-0.2650	-0.7504	0.5462	19.1631
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5837	-0.0052	0.0736	0.1921	-0.3648	-16.1537
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.3436	0.3473	0.0673	0.1740	-0.3539	-18.9863
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2727	8.3180	-0.4271	-1.2366	0.5475	13.5671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2727	8.3180	-0.4271	-1.2366	0.5475	13.5671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2748	4.7374	-0.1732	-0.5297	-0.1358	-12.9205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0947	5.0017	-0.1779	-0.5432	-0.1276	-15.0449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3520	4.1214	-0.2187	-0.6214	0.4216	14.3065
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3520	4.1214	-0.2187	-0.6214	0.4216	14.3065
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1955	0.5408	0.0353	0.0855	-0.2617	-12.1812
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0153	0.8051	0.0305	0.0719	-0.2535	-14.3056

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8251	3.8975	-0.2331	-0.6567	0.5271	19.2685
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8251	3.8975	-0.2331	-0.6567	0.5271	19.2685
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5715	-0.8766	0.1055	0.2858	-0.3838	-16.0483
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3313	-0.5242	0.0991	0.2678	-0.3730	-18.8809

### 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	13.7275
Shear X	-3.0724
Shear Z	-0.7013
Moment X	-2.0507
Moment Y (Twist)	0.9803
Moment Z	32.5243

### 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	8.3180
Shear X	-1.8251
Shear Z	-0.4271
Moment X	-1.2366
Moment Y (Twist)	0.5475
Moment Z	19.2685

## Project Details

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

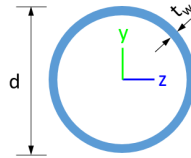


## Design Input Information

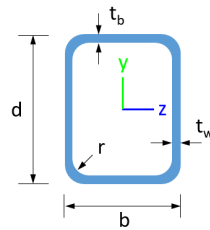
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

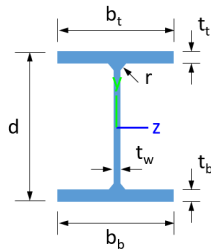
### Section Dimensions



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				
9	8in Pipe Sch 40	8.63	0.32				



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> )
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------







212	251.01	248.88	27.10	27.10	75.30	75.30
213	159.30	97.43	32.07	6.46	56.26	44.91
214	159.30	97.43	32.19	6.46	56.26	44.91
215	159.30	75.13	20.37	6.46	56.26	44.91
216	159.30	75.13	20.46	6.46	56.26	44.91
301	377.97	215.58	83.29	83.29	113.39	113.39
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	113.66	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	113.66	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	35.56	6.46	56.26	44.91
314	159.30	97.43	37.15	6.46	56.26	44.91
315	159.30	75.13	20.86	6.46	56.26	44.91
316	159.30	75.13	20.68	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	0.064	0.391	0.061	0.027	0.006	0.426	#13	0.438	Not Required	Pass
2	0.003	0.295	0.099	0.071	0.019	0.375	#21	0.054	Not Required	Pass
3	0.008	0.463	0.032	0.045	0.004	0.484	#21	0.046	Not Required	Pass
4	0.007	0.456	0.096	0.046	0.025	0.551	#21	0.082	Not Required	Pass
5	0.007	0.287	0.074	0.046	0.020	0.305	#21	0.076	Not Required	Pass
6	0.014	0.631	0.136	0.064	0.037	0.774	#21	0.046	Not Required	Pass
7	0.015	0.391	0.236	0.062	0.059	0.447	#21	0.076	Not Required	Pass
8	0.005	0.119	0.243	0.036	0.028	0.275	#24	0.102	Not Required	Pass
9	0.010	0.043	0.098	0.002	0.006	0.137	#21	0.206	Not Required	Pass
10	0.015	0.604	0.213	0.061	0.046	0.729	#21	0.082	Not Required	Pass
11	0.008	0.115	0.255	0.038	0.028	0.283	#24	0.102	Not Required	Pass
12	0.001	0.483	0.135	0.107	0.023	0.589	#21	0.054	Not Required	Pass
13	0.012	0.103	0.649	0.050	0.036	0.700	#21	0.306	Not Required	Pass
14	0.007	0.096	0.632	0.048	0.036	0.676	#24	0.204	Not Required	Pass
15	0.000	0.016	0.064	0.014	0.009	0.080	#21	Not Required	Not Required	Pass
16	0.000	0.016	0.064	0.014	0.009	0.080	#21	Not Required	Not Required	Pass
101	0.084	0.522	0.004	0.036	0.000	0.552	#13	0.438	Not Required	Pass
102	0.005	0.524	0.162	0.118	0.027	0.667	#21	0.036	Not Required	Pass
103	0.014	0.731	0.086	0.073	0.011	0.824	#21	0.046	Not Required	Pass
104	0.014	0.733	0.235	0.073	0.051	0.895	#21	0.082	Not Required	Pass
105	0.014	0.454	0.247	0.072	0.064	0.518	#21	0.076	Not Required	Pass
106	0.014	0.732	0.083	0.073	0.011	0.819	#21	0.046	Not Required	Pass
107	0.014	0.455	0.238	0.072	0.062	0.518	#21	0.076	Not Required	Pass
108	0.005	0.043	0.243	0.041	0.028	0.287	#21	0.102	Not Required	Pass
109	0.023	0.054	0.056	0.001	0.000	0.119	#21	0.206	Not Required	Pass
110	0.014	0.722	0.228	0.072	0.050	0.881	#21	0.082	Not Required	Pass

111	0.008	0.056	0.252	0.042	0.028	0.286	#21	0.102	Not Required	Pass
112	0.005	0.518	0.164	0.117	0.028	0.661	#21	0.036	Not Required	Pass
113	0.012	0.209	0.670	0.057	0.037	0.847	#21	0.306	Not Required	Pass
114	0.010	0.235	0.659	0.058	0.037	0.858	#21	0.306	Not Required	Pass
115	0.015	0.359	0.357	0.045	0.028	0.723	#21	0.507	Not Required	Pass
116	0.005	0.343	0.351	0.046	0.029	0.696	#21	0.507	Not Required	Pass
201	0.084	0.522	0.004	0.036	0.000	0.552	#13	0.438	Not Required	Pass
202	0.005	0.518	0.164	0.117	0.028	0.661	#21	0.036	Not Required	Pass
203	0.014	0.732	0.083	0.073	0.011	0.819	#21	0.046	Not Required	Pass
204	0.014	0.722	0.228	0.072	0.050	0.881	#21	0.082	Not Required	Pass
205	0.014	0.455	0.238	0.072	0.062	0.518	#21	0.076	Not Required	Pass
206	0.014	0.731	0.086	0.073	0.011	0.824	#21	0.046	Not Required	Pass
207	0.014	0.454	0.247	0.072	0.064	0.518	#21	0.076	Not Required	Pass
208	0.005	0.070	0.263	0.046	0.029	0.291	#21	0.102	Not Required	Pass
209	0.023	0.054	0.056	0.001	0.000	0.119	#21	0.206	Not Required	Pass
210	0.014	0.733	0.235	0.073	0.051	0.895	#21	0.082	Not Required	Pass
211	0.008	0.082	0.272	0.045	0.028	0.287	#21	0.102	Not Required	Pass
212	0.005	0.524	0.162	0.118	0.027	0.667	#21	0.036	Not Required	Pass
213	0.012	0.209	0.670	0.057	0.037	0.847	#21	0.306	Not Required	Pass
214	0.010	0.235	0.659	0.058	0.037	0.858	#21	0.306	Not Required	Pass
215	0.015	0.262	0.357	0.042	0.028	0.626	#21	0.507	Not Required	Pass
216	0.005	0.227	0.350	0.041	0.028	0.578	#21	0.507	Not Required	Pass
301	0.064	0.391	0.061	0.027	0.006	0.426	#13	0.438	Not Required	Pass
302	0.001	0.483	0.135	0.107	0.023	0.589	#21	0.054	Not Required	Pass
303	0.014	0.631	0.136	0.064	0.037	0.774	#21	0.046	Not Required	Pass
304	0.015	0.604	0.213	0.061	0.046	0.729	#21	0.082	Not Required	Pass
305	0.015	0.391	0.236	0.062	0.059	0.447	#21	0.076	Not Required	Pass
306	0.008	0.463	0.032	0.045	0.004	0.484	#21	0.046	Not Required	Pass
307	0.007	0.287	0.074	0.046	0.019	0.305	#21	0.076	Not Required	Pass
308	0.000	0.016	0.064	0.014	0.009	0.080	#21	Not Required	Not Required	Pass
309	0.010	0.043	0.098	0.002	0.006	0.137	#21	0.206	Not Required	Pass
310	0.007	0.456	0.096	0.046	0.025	0.551	#21	0.082	Not Required	Pass
311	0.000	0.016	0.064	0.014	0.009	0.080	#21	Not Required	Not Required	Pass
312	0.003	0.295	0.099	0.071	0.019	0.375	#21	0.054	Not Required	Pass
313	0.012	0.103	0.648	0.050	0.036	0.700	#21	0.204	Not Required	Pass
314	0.007	0.096	0.632	0.048	0.036	0.677	#24	0.306	Not Required	Pass
315	0.015	0.374	0.357	0.038	0.028	0.735	#21	0.507	Not Required	Pass
316	0.005	0.367	0.349	0.036	0.028	0.710	#21	0.507	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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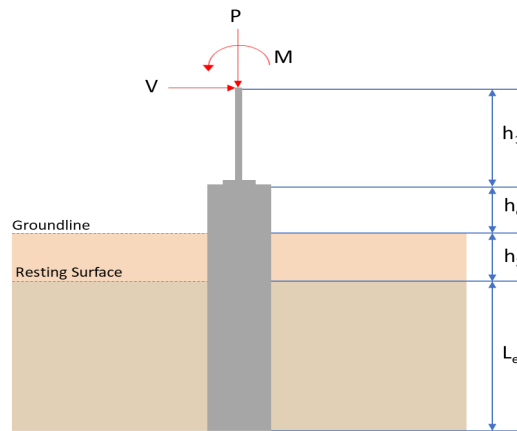
## SkyCiv Foundation Design

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)	8.318	13.728
$V_x$ (kip)	-1.825	-3.072
$V_z$ (kip)	0.427	0.701
$M_x$ (kipft)	1.236	2.049
$M_z$ (kipft)	19.269	32.524

### 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,

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

$$H_o = \frac{(-1.825 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.29061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(19.269 \text{ kipft}) + ((-1.825 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0683 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3408 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.427 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.067994 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.236 \text{ kipft}) + ((0.427 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19682 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.0422 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.3408 \text{ ft}), (3.0422 \text{ ft})]$$

$$L_{e,req} = 5.341 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(5.341 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.92887$$

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

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

$$q = \frac{(8.318 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51988 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51988 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25994$$

Status: **PASS**  
Ratio: **0.260**

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:**

$H_o = -0.29061 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.0683 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.0683 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.0683 \text{ kipft/ft})) + (4 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.961 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0683 \text{ kipft/ft})) + (3 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.0683 \text{ kipft/ft})) + (2 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20395 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.0683 \text{ kipft/ft})) + ((-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.8104 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.961 \text{ ft})}{2}$$

$$p_a = 0.29707 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20395 \text{ kip/ft}^2)}{(0.29707 \text{ kip/ft}^2)}$$

$$Ratio = 0.68652$$

$p_s$  - 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

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

$$Ratio = \frac{(0.8104 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.9396$$

Status: **PASS**  
Ratio: **0.690**

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

#### Considering z-direction:

$H_o = 0.067994 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.19682 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.19682 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.067994 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.19682 \text{ kipft/ft})) + (4 \times (0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1063 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.19682 \text{ kipft/ft})) + (3 \times (0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.19682 \text{ kipft/ft})) + (2 \times (0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.063509 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.19682 \text{ kipft/ft})) + ((0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.14238 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1063 \text{ ft})}{2}$$

$$p_a = 0.30798 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.063509 \text{ kip/ft}^2)}{(0.30798 \text{ kip/ft}^2)}$$

$$Ratio = 0.20621$$

$p_s$  - 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

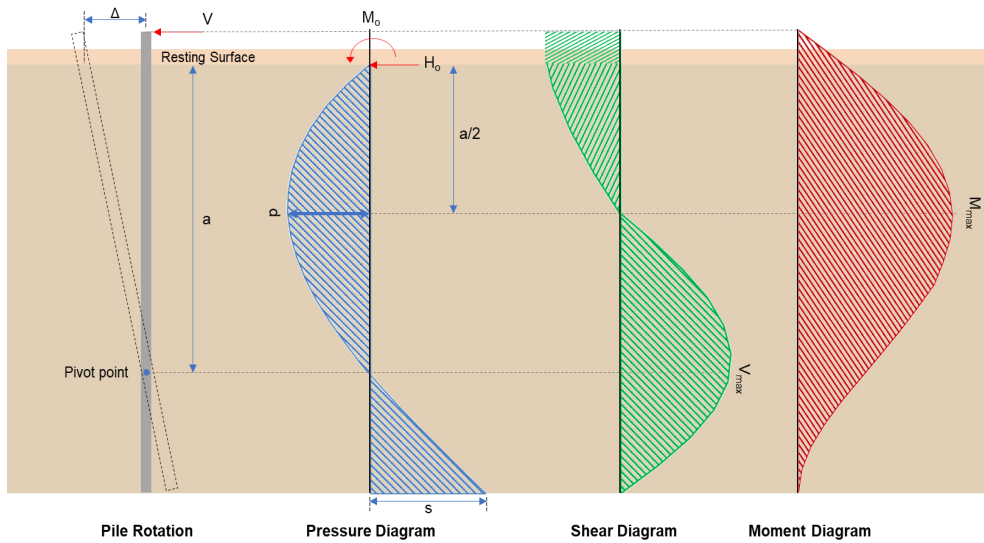
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.14238 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.16508$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.072 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.48917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(32.524 \text{ kipft}) + ((-3.072 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.179 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.179 \text{ kipft/ft})}{(-0.48917 \text{ kip/ft})}$$

$$E = 10.587 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.179 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.179 \text{ kipft/ft})) + (4 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (5.179 \text{ kipft/ft})) + (4 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9607 \text{ ft}$$

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

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

$$V_{max} = ((-0.48917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.6661 \text{ kip}$$

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

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

$$M_{max} = ((-0.48917 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(10.587 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.018 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.701 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.11162 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(2.049 \text{ kipft}) + ((0.701 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.32627 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.32627 \text{ kipft/ft})}{(0.11162 \text{ kip/ft})}$$

$$E = 2.923 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.32627 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.11162 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.32627 \text{ kipft/ft})) + (4 \times (0.11162 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1052 \text{ ft}$$

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

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

$$V_{max} = ((0.11162 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.923 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1052 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.923 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1052 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.69904 \text{ kip}$$

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

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

$$M_{max} = ((0.11162 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.923 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.923 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.923 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.7807 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.728 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.14 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.14 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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} = Minimum\ spacing\ of\ reinforcement,</math></p> $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625\ in))]$ $s_{rebar} = 1.5\ in$ <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}</math> - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625\ in)), (48 \times (0.375\ in)), Min((48\ in), (48\ in))]$ $s_{ties} = 10\ in$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5\ ksi) \times [(2304\ in^2) - (4.2951\ in^2)]) + ((60\ ksi) \times (4.2951\ in^2))]$ $\phi P_N = 2675.2\ kip$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.728\ kip)}{(2675.2\ kip)}$ $Ratio = 0.0051316$	<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\ in</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48\ in)$ $d = 38.4\ in$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4\ in)}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5\ ksi \rightarrow 2500\ psi</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ psi)} \times (48\ in) \times (38.4\ in)$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.728 \text{ kip} \rightarrow 13728 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13728 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.32 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.32 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.32 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.32 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.29 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.6661 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(7.6661 \text{ kip})}{(111.29 \text{ kip})}$$

$$Ratio = 0.068887$$

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

**Considering z-direction:**

$V_{max} = 0.69904 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.69904 \text{ kip})}{(111.29 \text{ kip})}$$

$$Ratio = 0.0062815$$

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

**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:**

$M_{max} = 21.018 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.018 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.084206$$

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

**Considering z-direction:**

$M_{max} = 1.7807 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.7807 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0071342$$

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

REFERENCES	CALCULATIONS	RESULTS
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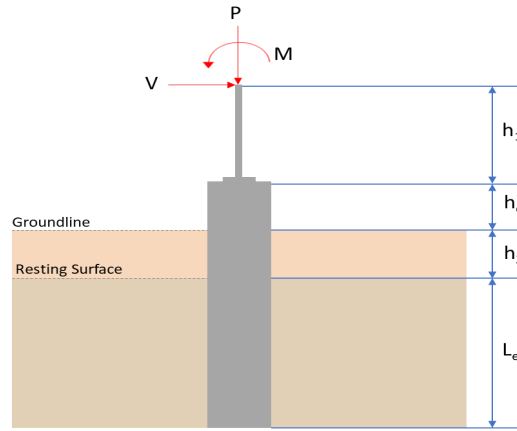
## SkyCiv Foundation Design

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)	8.318	13.728
$V_x$ (kip)	-1.825	-3.072
$V_z$ (kip)	-0.427	-0.701
$M_x$ (kipft)	-1.237	-2.051
$M_z$ (kipft)	19.269	32.524

### 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,

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

$$H_o = \frac{(-1.825 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.29061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(19.269 \text{ kipft}) + ((-1.825 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0683 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.3408 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.427 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.067994 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.237 \text{ kipft}) + ((-0.427 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19697 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9748 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(5.3408 \text{ ft}), (1.9748 \text{ ft})]$$

$$L_{e,req} = 5.341 \text{ ft}$$

$L_e$  - 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

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

$$Ratio = \frac{(5.341 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.92887$$

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

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

$$q = \frac{(8.318 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51988 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51988 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25994$$

Status: **PASS**  
Ratio: **0.260**

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:**

$H_o = -0.29061 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.0683 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.0683 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.0683 \text{ kipft/ft})) + (4 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.961 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.0683 \text{ kipft/ft})) + (3 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.0683 \text{ kipft/ft})) + (2 \times (-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.20395 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.0683 \text{ kipft/ft})) + ((-0.29061 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.8104 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.961 \text{ ft})}{2}$$

$$p_a = 0.29707 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20395 \text{ kip/ft}^2)}{(0.29707 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.68652$$

$p_s$  - 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

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

$$\text{Ratio} = \frac{(0.8104 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9396$$

Status: **PASS**  
Ratio: **0.690**

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

#### Considering z-direction:

$H_o = -0.067994 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.19697 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.19697 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.067994 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.19697 \text{ kipft/ft})) + (4 \times (-0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1062 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.19697 \text{ kipft/ft})) + (3 \times (-0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.19697 \text{ kipft/ft})) + (2 \times (-0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = -0.017603 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.19697 \text{ kipft/ft})) + ((-0.067994 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.00054182 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1062 \text{ ft})}{2}$$

$$p_a = 0.30797 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.017603 \text{ kip/ft}^2)}{(0.30797 \text{ kip/ft}^2)}$$

$$Ratio = -0.057159$$

$p_s$  - 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

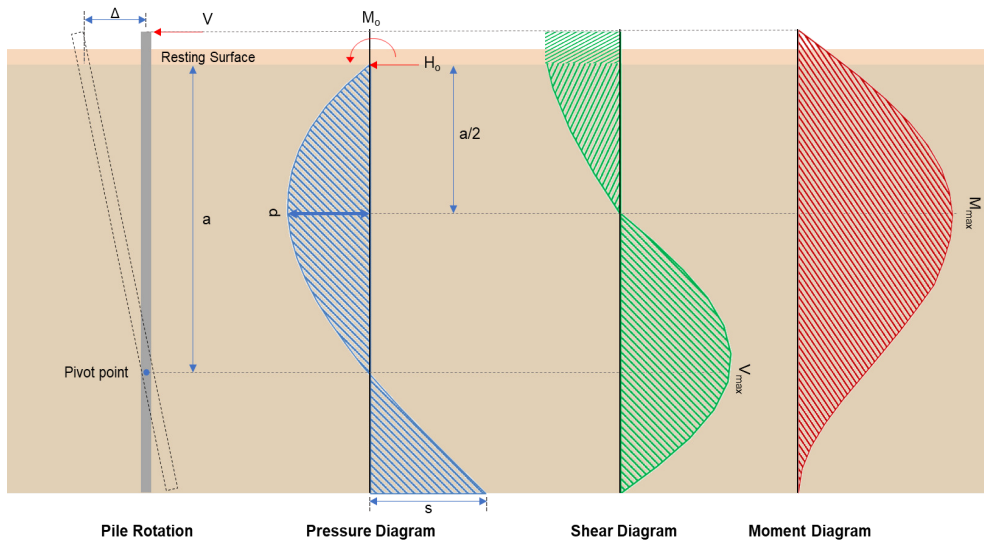
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00054182 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0006282$$

Status: **PASS**  
Ratio: **-0.060**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.072 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.48917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(32.524 \text{ kipft}) + ((-3.072 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.179 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.179 \text{ kipft/ft})}{(-0.48917 \text{ kip/ft})}$$

$$E = 10.587 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.179 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.179 \text{ kipft/ft})) + (4 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (5.179 \text{ kipft/ft})) + (4 \times (-0.48917 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9607 \text{ ft}$$

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

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

$$V_{max} = ((-0.48917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9607 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.6661 \text{ kip}$$

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

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

$$M_{max} = ((-0.48917 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(10.587 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.587 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.9607 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.018 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.701 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11162 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(2.051 \text{ kipft}) + ((-0.701 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.32659 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.32659 \text{ kipft/ft})}{(-0.11162 \text{ kip/ft})}$$

$$E = 2.9258 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.32659 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.11162 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.32659 \text{ kipft/ft})) + (4 \times (-0.11162 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1051 \text{ ft}$$

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

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

$$V_{max} = ((-0.11162 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.9258 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1051 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.9258 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1051 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.69943 \text{ kip}$$

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

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

$$M_{max} = ((-0.11162 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[ \left( \frac{(2.9258 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1051 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.9258 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.1051 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.9258 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.1051 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.7818 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.728 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.14 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.14 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{Max}[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = \text{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} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]</math></p> <p><math>s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{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><i>Ratio - Capacity</i></p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(13.728 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0051316</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 = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = \text{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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.728 \text{ kip} \rightarrow 13728 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13728 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.32 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.32 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.32 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.32 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.29 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 7.6661 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(7.6661 \text{ kip})}{(111.29 \text{ kip})}$$

$$Ratio = 0.068887$$

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

**Considering z-direction:**

$V_{max} = 0.69943 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.69943 \text{ kip})}{(111.29 \text{ kip})}$$

$$Ratio = 0.006285$$

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

**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:**

$M_{max} = 21.018 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.018 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.084206$$

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

**Considering z-direction:**

$M_{max} = 1.7818 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.7818 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0071387$$

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

REFERENCES	CALCULATIONS	RESULTS
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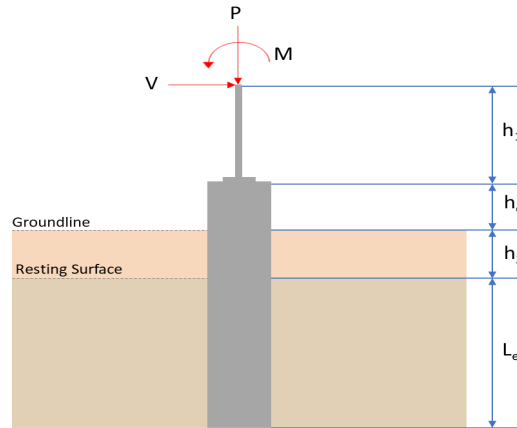
## SkyCiv Foundation Design

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

$L = 6.25$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	10.926	18.093
$V_x$ (kip)	-2.448	-4.116
$V_z$ (kip)	-0.030	-0.048
$M_x$ (kipft)	-0.085	-0.139
$M_z$ (kipft)	25.407	43.503

### 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,

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

$$H_o = \frac{(-2.448 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.38981 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(25.407 \text{ kipft}) + ((-2.448 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0457 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.7423 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0047771 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.085 \text{ kipft}) + ((-0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013535 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.93419 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.7423 \text{ ft}), (0.93419 \text{ ft})]$$

$$L_{e,req} = 5.742 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.742 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.91872$$

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

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(10.926 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.68288 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.68288 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.34144$$

Status: **PASS**  
Ratio: **0.340**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.38981 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0457 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.0457 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0457 \text{ kipft/ft})) + (4 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3159 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0457 \text{ kipft/ft})) + (3 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0457 \text{ kipft/ft})) + (2 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20812 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.0457 \text{ kipft/ft})) + ((-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86862 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3159 \text{ ft})}{2}$$

$$p_a = 0.32369 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20812 \text{ kip/ft}^2)}{(0.32369 \text{ kip/ft}^2)}$$

$$Ratio = 0.64298$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.86862 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.92653$$

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

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

#### Considering z-direction:

$H_o = -0.0047771 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.013535 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.013535 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.013535 \text{ kipft/ft})) + (4 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4767 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.013535 \text{ kipft/ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.013535 \text{ kipft/ft})) + (2 \times (-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0012613 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.013535 \text{ kipft/ft})) + ((-0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.00042803 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4767 \text{ ft})}{2}$$

$$p_a = 0.33575 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0012613 \text{ kip/ft}^2)}{(0.33575 \text{ kip/ft}^2)}$$

$$Ratio = -0.0037567$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

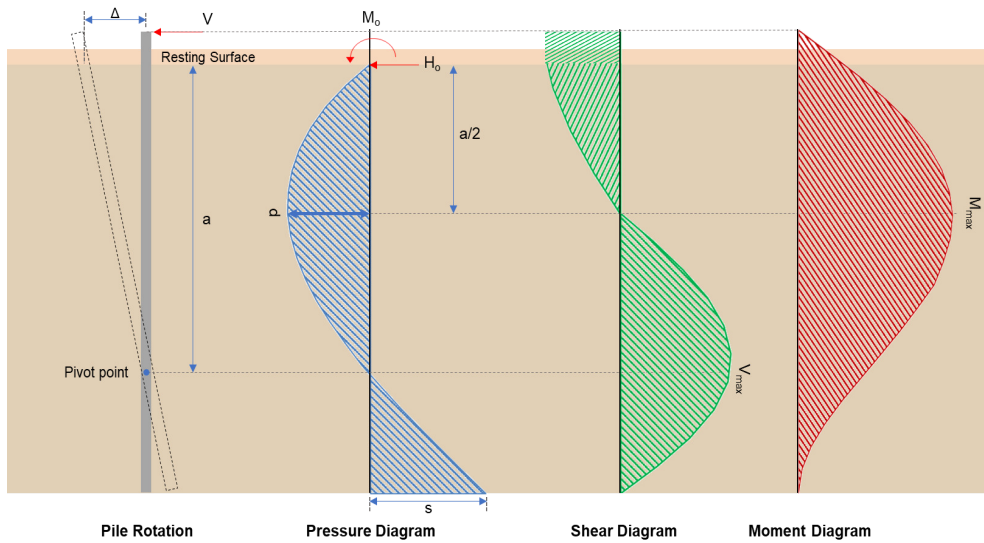
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00042803 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = -0.00045656$$

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,

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

$$H_o = \frac{(-4.116 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.65541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(43.503 \text{ kipft}) + ((-4.116 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.9272 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.9272 \text{ kipft/ft})}{(-0.65541 \text{ kip/ft})}$$

$$E = 10.569 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.9272 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.65541 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times 6.9272) + (4 \times (-0.65541) \times 6.25)}$$

$$a = \frac{(-0.65541 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.9272 \text{ kipft/ft})) + (4 \times (-0.65541 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3139 \text{ ft}$$

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

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

$$V_{max} = ((-0.65541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3139 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3139 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.574 \text{ kip}$$

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

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

$$M_{max} = ((-0.65541 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(10.569 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.431 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.048 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0076433 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.139 \text{ kipft}) + ((-0.048 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022134 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.022134 \text{ kipft/ft})}{(-0.0076433 \text{ kip/ft})}$$

$$E = 2.8958 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.022134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0076433 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.022134 \text{ kipft/ft})) + (4 \times (-0.0076433 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4739 \text{ ft}$$

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

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

$$V_{max} = ((-0.0076433 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4739 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4739 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.04546 \text{ kip}$$

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

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

$$M_{max} = ((-0.0076433 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(2.8958 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12504 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.093 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.995 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.995 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(18.093 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0067633</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.093 \text{ kip} \rightarrow 18093 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18093 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.9 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.9 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.9 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.9 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.66 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 9.574 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(9.574 \text{ kip})}{(111.66 \text{ kip})}$$

$$Ratio = 0.085739$$

**Considering z-direction:**

$V_{max} = 0.04546 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.04546 \text{ kip})}{(111.66 \text{ kip})}$$

$$Ratio = 0.00040711$$

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

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 28.431 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(28.431 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11391$$

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

**Considering z-direction:**

$M_{max} = 0.12504 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(0.12504 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00050094$$

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

REFERENCES	CALCULATIONS	RESULTS
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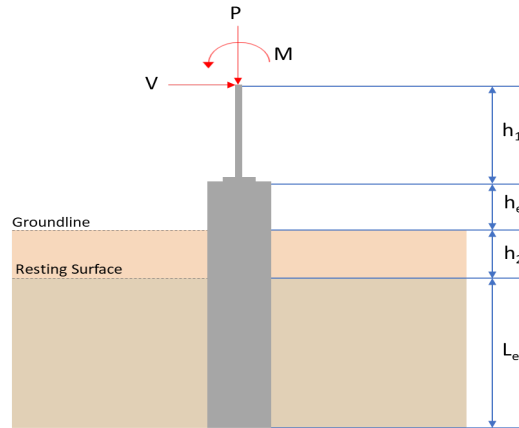
## SkyCiv Foundation Design

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

$L = 6.25$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	10.926	18.093
$V_x$ (kip)	-2.448	-4.116
$V_z$ (kip)	0.030	0.048
$M_x$ (kipft)	0.085	0.139
$M_z$ (kipft)	25.407	43.503

### 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,

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

$$H_o = \frac{(-2.448 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.38981 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(25.407 \text{ kipft}) + ((-2.448 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0457 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.7423 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0047771 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.085 \text{ kipft}) + ((0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013535 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1199 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(5.7423 \text{ ft}), (1.1199 \text{ ft})]$$

$$L_{e,req} = 5.742 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(5.742 \text{ ft})}{(6.25 \text{ ft})}$$

$$Ratio = 0.91872$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(10.926 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.68288 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.68288 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.34144$$

Status: **PASS**  
Ratio: **0.340**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.38981 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0457 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.0457 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.0457 \text{ kipft/ft})) + (4 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3159 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0457 \text{ kipft/ft})) + (3 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.0457 \text{ kipft/ft})) + (2 \times (-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20812 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.0457 \text{ kipft/ft})) + ((-0.38981 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86862 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3159 \text{ ft})}{2}$$

$$p_a = 0.32369 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20812 \text{ kip/ft}^2)}{(0.32369 \text{ kip/ft}^2)}$$

$$Ratio = 0.64298$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.86862 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.92653$$

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

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

#### Considering z-direction:

$H_o = 0.0047771 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.013535 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.013535 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.013535 \text{ kipft/ft})) + (4 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4767 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.013535 \text{ kipft/ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.013535 \text{ kipft/ft})) + (2 \times (0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0039527 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.013535 \text{ kipft/ft})) + ((0.0047771 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0087439 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4767 \text{ ft})}{2}$$

$$p_a = 0.33575 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0039527 \text{ kip/ft}^2)}{(0.33575 \text{ kip/ft}^2)}$$

$$Ratio = 0.011773$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

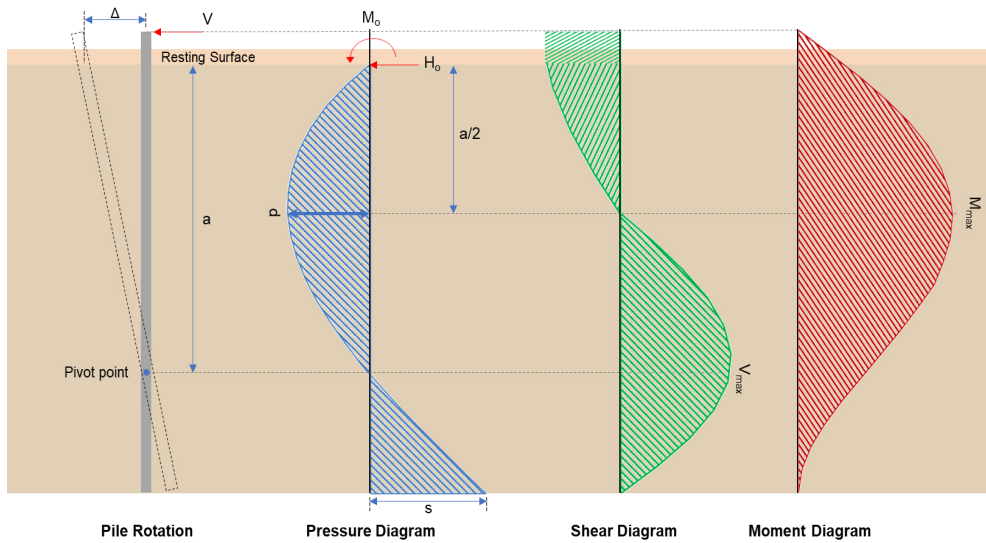
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0087439 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0093269$$

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,

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

$$H_o = \frac{(-4.116 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.65541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(43.503 \text{ kipft}) + ((-4.116 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.9272 \text{ kipft/ft}$$

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

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

$$E = \frac{(6.9272 \text{ kipft/ft})}{(-0.65541 \text{ kip/ft})}$$

$$E = 10.569 \text{ ft}$$

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

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

$$a = \frac{(4 \times (6.9272 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.65541 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.9272 \text{ kipft/ft})) + (4 \times (-0.65541 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.65541 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.9272 \text{ kipft/ft})) + (4 \times (-0.65541 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3139 \text{ ft}$$

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

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

$$V_{max} = ((-0.65541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3139 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3139 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.574 \text{ kip}$$

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

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

$$M_{max} = ((-0.65541 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(10.569 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.569 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.3139 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.431 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.048 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0076433 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.139 \text{ kipft}) + ((0.048 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022134 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.022134 \text{ kipft/ft})}{(0.0076433 \text{ kip/ft})}$$

$$E = 2.8958 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.022134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0076433 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.022134 \text{ kipft/ft})) + (4 \times (0.0076433 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4739 \text{ ft}$$

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

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

$$V_{max} = ((0.0076433 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4739 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4739 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.04546 \text{ kip}$$

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

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

$$M_{max} = ((0.0076433 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[ \left( \frac{(2.8958 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left( \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.8958 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left( \frac{(4.4739 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.12504 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(18.093 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.995 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.995 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(18.093 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0067633</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 18.093 \text{ kip} \rightarrow 18093 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18093 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.9 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.9 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.9 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.9 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.66 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 9.574 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(9.574 \text{ kip})}{(111.66 \text{ kip})}$$

$$Ratio = 0.085739$$

**Considering z-direction:**

$V_{max} = 0.04546 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.04546 \text{ kip})}{(111.66 \text{ kip})}$$

$$Ratio = 0.00040711$$

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

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:**

$M_{max} = 28.431 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(28.431 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11391$$

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

**Considering z-direction:**

$M_{max} = 0.12504 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$\text{Ratio} = \frac{(0.12504 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00050094$$

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