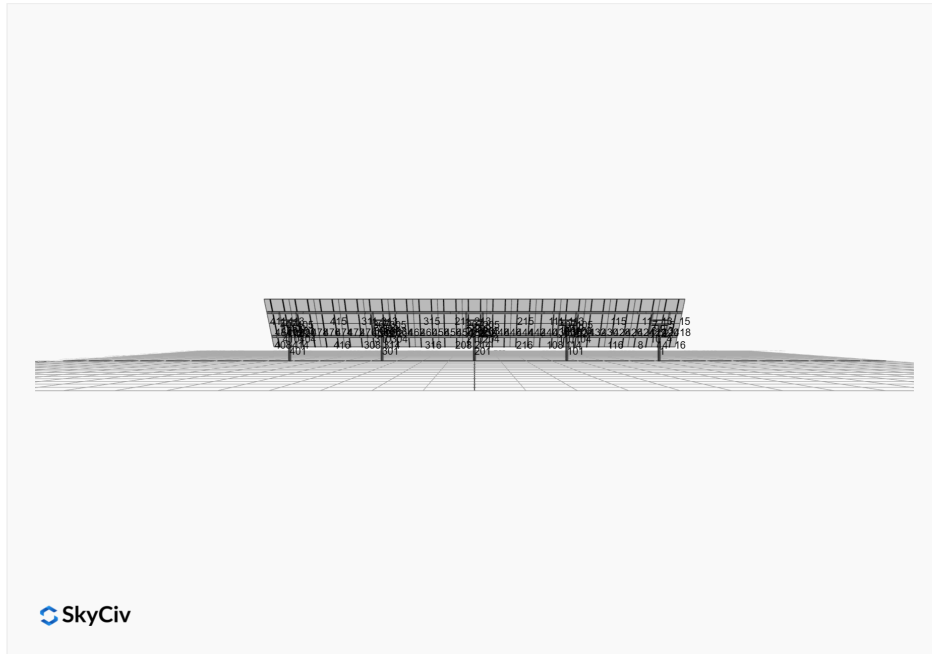


# Project Details



**Project Name:** Reebel Project **Date:** Tue May 13 2025  
**Location:** 4729 Durst Clagg Rd, Cortland, OH 44410, USA **Number of Modules:** 68  
**Unique ID:** 5P-22.5-6TOP-SD-12-L-4Hx17W-9D3B **Number of Poles:** 5  
**Dealer:** \_\_\_\_\_ **Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	15.05 ft
<b>Array Dimensions E/W</b>	100.24 ft
<b>Winter Tilt Angle</b>	50
<b>Front Edge Clearance</b>	3 ft

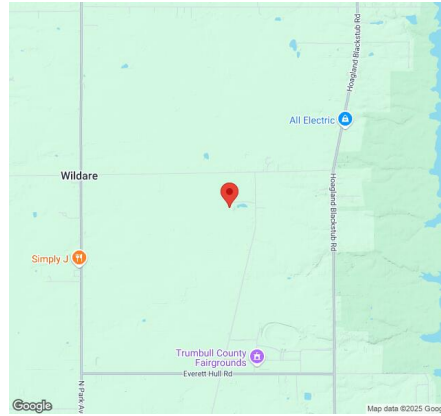
## MT Solar Bill of Materials (5P-22.5-6TOP-SD-12-L-4Hx17W-9D3B)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-SD	H-Frame Assembly-SD	5
MTS-SD-Wing-12	12IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	16
MTS-CLAMP-HOOK-4PK	Hook Clamp	17

## Rail Bill of Materials

Part	Qty
Rails (181in)	34
Rail Attachment	68
Module Mid Clamp	102
Module End Clamp	68
Ground Lug	17

## Site Details:



**Site Address:** 4729 Durst Clagg Rd, Cortland, OH 44410, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	44.65 in
<b>Module Length:</b>	69.76in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	17
<b>Total Number of Modules:</b>	68
<b>Winter Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	3
<b>Total Array Height at Tilt:</b>	14.53 ft
<b>Total Frame Length:</b>	99.50 ft
<b>Module Info/Notes:</b>	SEG 405
<b>Array Dimensions N/S:</b>	15.05 ft
<b>Array Dimensions E/W:</b>	100.24 ft
<b>Rail Length:</b>	180.60 in
<b>Rail Spacing:</b>	2.95 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 80
<b>Pole Length above Grade:</b>	8.76 ft
<b>Number of Poles:</b>	5
<b>Pole Spacing:</b>	22.5 ft

### Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 7.75 ft Pile 2: 8.50 ft Pile 3: 8.50 ft Pile 4: 8.50 ft Pile 5: 7.75 ft
<b>Foundation Volume:</b>	10.734 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	4729 Durst Clagg Rd, Cortland, OH 44410, USA
<b>Wind Speed:</b>	103 mph

**Snow Load:**

20 psf

### **Design Disclaimer**

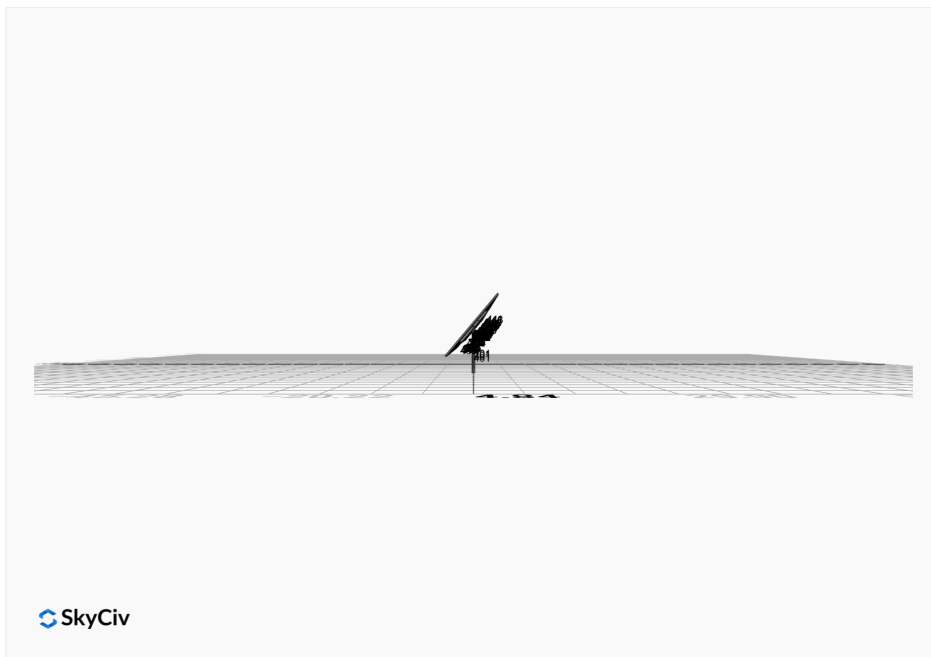
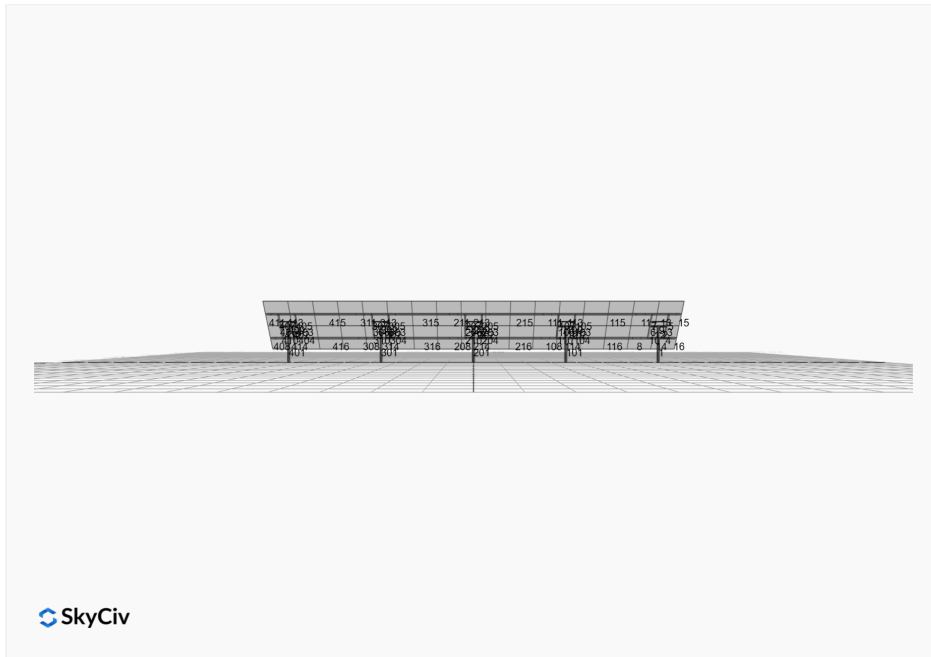
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

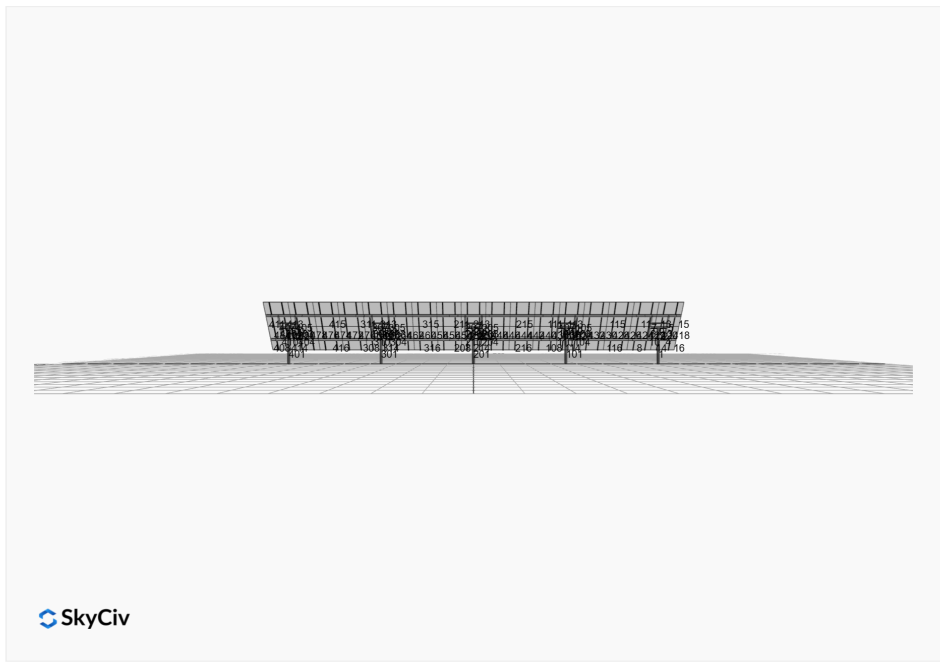
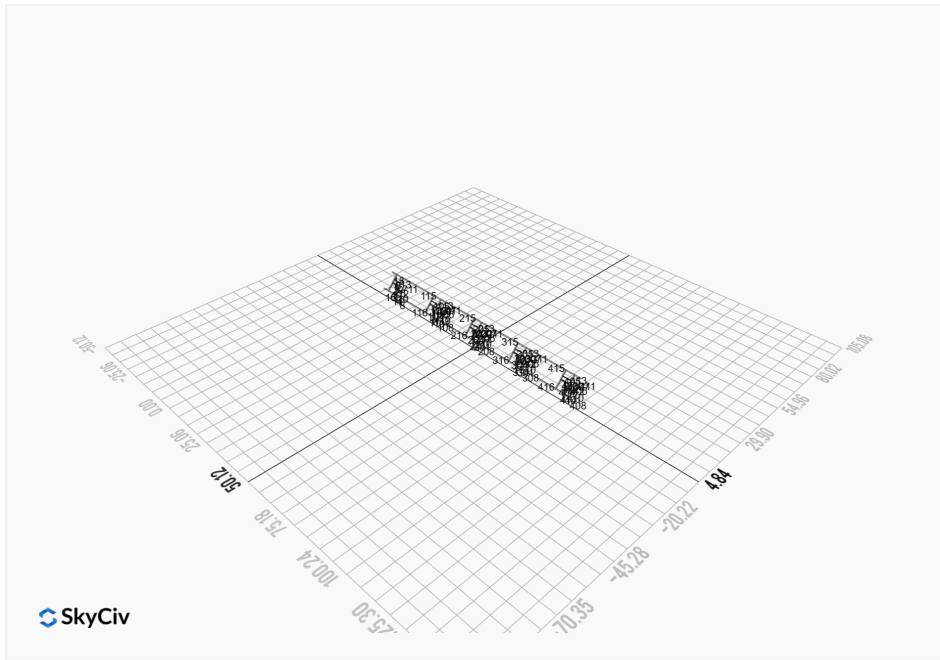
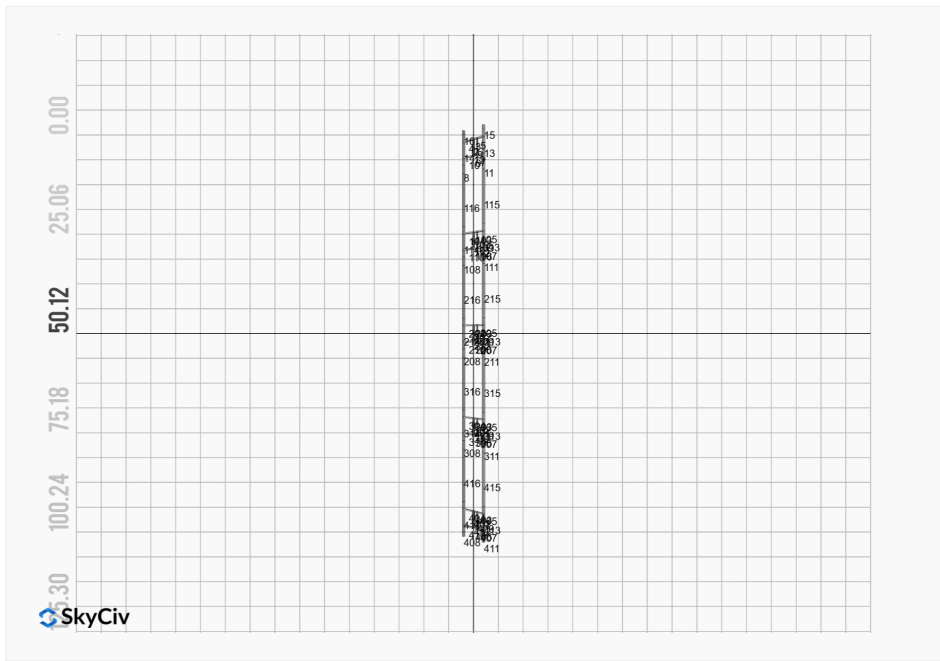
## AutoDesigner Input

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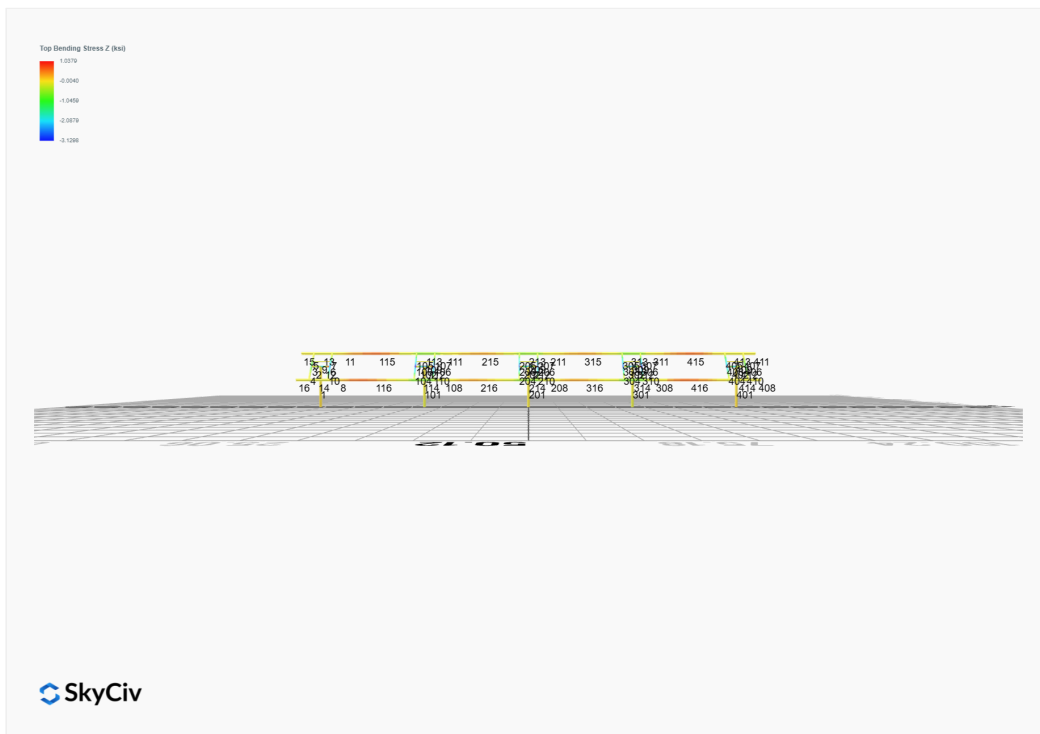
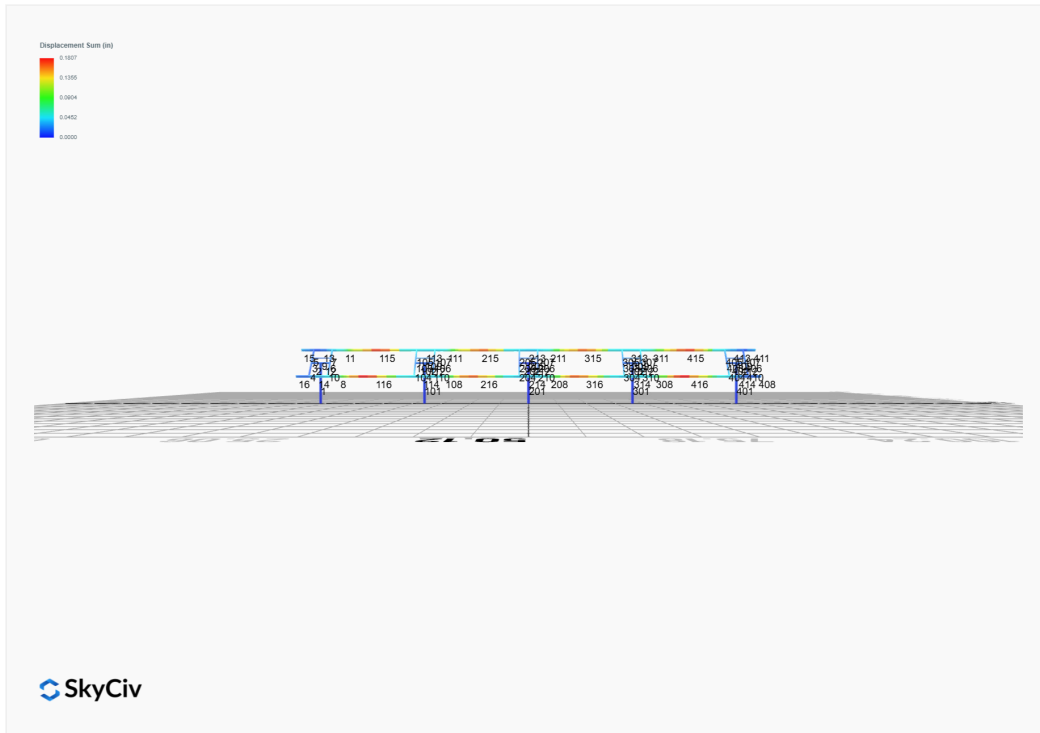
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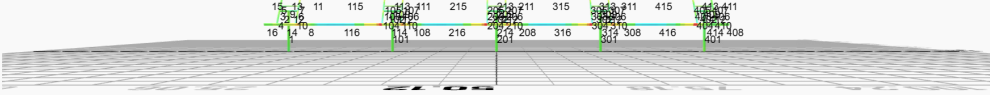
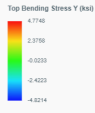
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



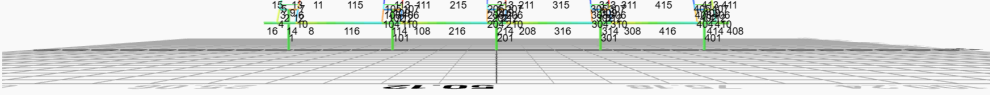
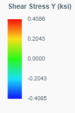


# FEM Results (Envelope Worst Case for each member)

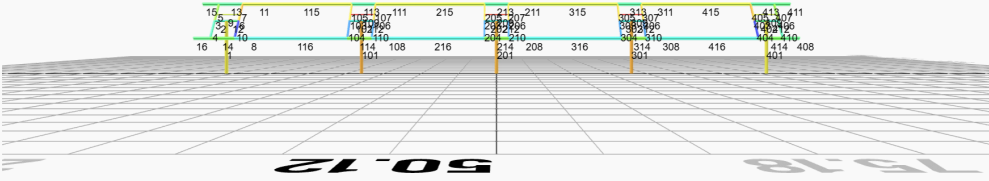
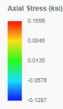




SkyCiv



SkyCiv



## Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 2. D + L	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 3. D + (S or Lr or R)	0.0629	2.4509	0.2404	0.5773	-0.3399	-0.5091
ULS: 3. D + (S or Lr or R)	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0578	2.2852	0.2210	0.5307	-0.3124	-0.4669
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 5b. D + 0.7E	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0578	2.2852	0.2210	0.5307	-0.3124	-0.4669
ULS: 8. 0.6D + 0.7E	0.0255	1.0727	0.0976	0.2345	-0.1380	-0.2040
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9919	3.4660	0.5334	1.2072	-1.7168	17.8361
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0759	0.1107	-0.2025	-0.4131	1.2370	-18.1761
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4680	3.5437	0.4990	1.1430	-1.4275	13.1653
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0578	2.2852	0.2210	0.5307	-0.3124	-0.4669
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5828	1.0273	-0.0529	-0.0722	0.7879	-13.8439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0578	2.2852	0.2210	0.5307	-0.3124	-0.4669
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4833	3.0464	0.4407	1.0031	-1.3451	13.2921
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5676	0.5300	-0.1112	-0.2121	0.8703	-13.7171
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0425	1.7879	0.1627	0.3908	-0.2300	-0.3401
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0089	2.7508	0.4683	1.0509	-1.6248	17.9721
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0255	1.0727	0.0976	0.2345	-0.1380	-0.2040
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0589	-0.6044	-0.2676	-0.5694	1.3290	-18.0401
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0255	1.0727	0.0976	0.2345	-0.1380	-0.2040

### Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	5.2736
Shear X	-3.4505
Shear Z	0.8556
Moment X	1.9318
Moment Y (Twist)	2.8204
Moment Z	30.4808

### 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	3.5437
Shear X	-2.0759
Shear Z	0.5334
Moment X	1.2072
Moment Y (Twist)	1.7168
Moment Z	18.1761

## 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	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 2. D + L	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 3. D + (S or Lr or R)	-0.0687	3.4240	-0.0269	-0.0684	0.0810	0.5745
ULS: 3. D + (S or Lr or R)	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0631	3.1796	-0.0247	-0.0629	0.0744	0.5293

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 5b. D + 0.7E	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0631	3.1796	-0.0247	-0.0629	0.0744	0.5293
ULS: 8. 0.6D + 0.7E	-0.0279	1.4679	-0.0109	-0.0278	0.0329	0.2363
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1486	5.0799	-0.0461	-0.1232	0.1039	27.7001
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0572	-0.1884	0.0109	0.0326	-0.0007	-26.2030
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3897	5.1547	-0.0456	-0.1205	0.1113	21.0091
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0631	3.1796	-0.0247	-0.0629	0.0744	0.5293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2646	1.2035	-0.0028	-0.0037	0.0328	-19.4183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0631	3.1796	-0.0247	-0.0629	0.0744	0.5293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3731	4.4215	-0.0391	-0.1040	0.0916	20.8735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2813	0.4704	0.0037	0.0129	0.0131	-19.5538
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0465	2.4465	-0.0182	-0.0463	0.0548	0.3938
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1300	4.1013	-0.0388	-0.1046	0.0820	27.5426
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0279	1.4679	-0.0109	-0.0278	0.0329	0.2363
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0758	-1.1670	0.0182	0.0512	-0.0227	-26.3605
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0279	1.4679	-0.0109	-0.0278	0.0329	0.2363

### 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	7.8131
Shear X	-5.2359
Shear Z	-0.0720
Moment X	-0.1936
Moment Y (Twist)	0.1558
Moment Z	46.6409

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.1547
Shear X	-3.1486
Shear Z	-0.0461
Moment X	-0.1232
Moment Y (Twist)	0.1113
Moment Z	27.7001

### 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	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 2. D + L	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 3. D + (S or Lr or R)	0.0116	3.3479	-0.0001	-0.0017	0.0003	-0.0340
ULS: 3. D + (S or Lr or R)	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0107	3.1096	-0.0001	-0.0015	0.0002	-0.0300
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 5b. D + 0.7E	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0107	3.1096	-0.0001	-0.0015	0.0002	-0.0300
ULS: 8. 0.6D + 0.7E	0.0047	1.4370	-0.0000	-0.0007	0.0001	-0.0109
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9821	4.9009	-0.0004	-0.0035	-0.0010	26.6862
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9967	-0.1098	0.0003	0.0011	0.0013	-26.0112
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182

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	-2.2318	4.9891	-0.0003	-0.0033	-0.0006	19.9982
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0107	3.1096	-0.0001	-0.0015	0.0002	-0.0300
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2523	1.2311	0.0002	0.0002	0.0011	-19.5248
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0107	3.1096	-0.0001	-0.0015	0.0002	-0.0300
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.2346	4.2744	-0.0003	-0.0029	-0.0007	20.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2495	0.5164	0.0002	0.0006	0.0010	-19.5129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0078	2.3950	-0.0000	-0.0011	0.0002	-0.0182
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9852	3.9429	-0.0003	-0.0030	-0.0010	26.6934
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0047	1.4370	-0.0000	-0.0007	0.0001	-0.0109
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9935	-1.0678	0.0003	0.0016	0.0012	-26.0039
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0047	1.4370	-0.0000	-0.0007	0.0001	-0.0109

### 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	7.5282
Shear X	-4.9938
Shear Z	-0.0006
Moment X	-0.0056
Moment Y (Twist)	0.0021
Moment Z	45.0453

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.9891
Shear X	-2.9967
Shear Z	-0.0004
Moment X	-0.0035
Moment Y (Twist)	0.0013
Moment Z	26.6934

### 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	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 2. D + L	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 3. D + (S or Lr or R)	-0.0687	3.4240	0.0265	0.0646	-0.0801	0.5744
ULS: 3. D + (S or Lr or R)	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0631	3.1797	0.0243	0.0593	-0.0736	0.5292
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 5b. D + 0.7E	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0631	3.1797	0.0243	0.0593	-0.0736	0.5292
ULS: 8. 0.6D + 0.7E	-0.0279	1.4679	0.0108	0.0262	-0.0325	0.2362
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1486	5.0800	0.0449	0.1152	-0.1043	27.7000
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.0573	-0.1884	-0.0102	-0.0300	0.0024	-26.2032
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3897	5.1547	0.0446	0.1130	-0.1112	21.0090
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0631	3.1797	0.0243	0.0593	-0.0736	0.5292
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2647	1.2035	0.0032	0.0041	-0.0311	-19.4185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0631	3.1797	0.0243	0.0593	-0.0736	0.5292
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3731	4.4216	0.0381	0.0973	-0.0918	20.8734
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2813	0.4703	-0.0032	-0.0116	-0.0117	-19.5540
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0465	2.4465	0.0179	0.0437	-0.0542	0.3937

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1300	4.1013	0.0377	0.0977	-0.0827	27.5426
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0279	1.4679	0.0108	0.0262	-0.0325	0.2362
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.0759	-1.1670	-0.0174	-0.0475	0.0241	-26.3607
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0279	1.4679	0.0108	0.0262	-0.0325	0.2362

### 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	7.8132
Shear X	-5.2360
Shear Z	0.0700
Moment X	0.1807
Moment Y (Twist)	0.1566
Moment Z	46.6408

### Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	5.1547
Shear X	-3.1486
Shear Z	0.0449
Moment X	0.1152
Moment Y (Twist)	0.1112
Moment Z	27.7000

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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 2. D + L	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 3. D + (S or Lr or R)	0.0629	2.4507	-0.2400	-0.5793	0.3360	-0.5096
ULS: 3. D + (S or Lr or R)	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0578	2.2850	-0.2206	-0.5326	0.3088	-0.4673
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 5b. D + 0.7E	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0578	2.2850	-0.2206	-0.5326	0.3088	-0.4673
ULS: 8. 0.6D + 0.7E	0.0255	1.0726	-0.0975	-0.2353	0.1364	-0.2042
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9916	3.4656	-0.5318	-1.2089	1.7053	17.8336
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0757	0.1108	0.2015	0.4120	-1.2308	-18.1742
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4678	3.5434	-0.4976	-1.1451	1.4173	13.1632
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0578	2.2850	-0.2206	-0.5326	0.3088	-0.4673
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5827	1.0273	0.0524	0.0706	-0.7848	-13.8427
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0578	2.2850	-0.2206	-0.5326	0.3088	-0.4673
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4830	3.0462	-0.4395	-1.0047	1.3358	13.2901
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5674	0.5300	0.1105	0.2110	-0.8663	-13.7158
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0426	1.7877	-0.1624	-0.3922	0.2274	-0.3404
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0086	2.7505	-0.4669	-1.0520	1.6143	17.9697
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0255	1.0726	-0.0975	-0.2353	0.1364	-0.2042
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0586	-0.6043	0.2665	0.5689	-1.3218	-18.0381
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0255	1.0726	-0.0975	-0.2353	0.1364	-0.2042

### Worst Case Reactions LRFD

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

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

### Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	5.2731
Shear X	-3.4501
Shear Z	-0.8530
Moment X	-1.9343
Moment Y (Twist)	2.8018
Moment Z	30.4779

Result	Value (kip, kip-ft)
Axial	3.5434
Shear X	-2.0757
Shear Z	-0.5318
Moment X	-1.2089
Moment Y (Twist)	1.7053
Moment Z	18.1742

# Project Details

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



## Design Input Information

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

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

**Section Dimensions**

ID	Name	d (in)	$t_w$ (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
8	6in Pipe Sch 80	6.63	0.43					

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

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

Section Properties								
ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{y0}$ (in <sup>4</sup> )	$I_{z0}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{y0}$ (in <sup>3</sup> )	$S_{z0}$ (in <sup>3</sup> )







103	79.65	74.89	10.99	6.26	29.14	16.61
104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	140.22	16.17	16.17	42.85	42.85
113	120.60	84.03	18.16	6.45	30.09	45.74
114	120.60	84.03	18.09	6.45	30.09	45.74
115	120.60	20.58	10.78	6.45	30.09	45.74
116	120.60	20.58	11.74	6.45	30.09	45.74
201	378.22	180.39	62.23	62.23	113.47	113.47
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	10.99	6.26	29.14	16.61
208	120.60	115.40	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.84	10.99	6.26	29.14	16.61
211	120.60	115.40	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.03	18.07	6.45	30.09	45.74
214	120.60	84.03	18.08	6.45	30.09	45.74
215	120.60	20.58	11.54	6.45	30.09	45.74
216	120.60	20.58	11.80	6.45	30.09	45.74
301	378.22	180.39	62.23	62.23	113.47	113.47
302	142.83	140.22	16.17	16.17	42.85	42.85
303	79.65	74.89	10.99	6.26	29.14	16.61
304	79.65	72.84	10.99	6.26	29.14	16.61
305	79.65	74.30	10.99	6.26	29.14	16.61
306	79.65	74.89	10.99	6.26	29.14	16.61
307	79.65	74.30	10.99	6.26	29.14	16.61
308	120.60	115.40	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.84	10.99	6.26	29.14	16.61
311	120.60	115.40	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	84.03	18.16	6.45	30.09	45.74
314	120.60	84.03	18.09	6.45	30.09	45.74
315	120.60	20.58	11.12	6.45	30.09	45.74
316	120.60	20.58	11.86	6.45	30.09	45.74
401	378.22	180.39	62.23	62.23	113.47	113.47
402	142.83	140.22	16.17	16.17	42.85	42.85
403	79.65	74.89	10.99	6.26	29.14	16.61
404	79.65	72.84	10.99	6.26	29.14	16.61
405	79.65	74.30	10.99	6.26	29.14	16.61
406	79.65	74.89	10.99	6.26	29.14	16.61

407	79.65	74.30	10.99	6.26	29.14	16.61
408	120.60	113.97	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.84	10.99	6.26	29.14	16.61
411	120.60	113.97	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	84.02	20.86	6.45	30.09	45.74
414	120.60	84.02	19.86	6.45	30.09	45.74
415	120.60	20.58	10.95	6.45	30.09	45.74
416	120.60	20.58	10.78	6.45	30.09	45.74

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.029	0.490	0.089	0.030	0.008	0.529	#13	0.503	Not Required	Pass
2	0.004	0.080	0.112	0.030	0.028	0.192	#13	0.052	Not Required	Pass
3	0.004	0.305	0.035	0.028	0.011	0.336	#13	0.044	Not Required	Pass
4	0.004	0.310	0.123	0.031	0.023	0.434	#13	0.117	Not Required	Pass
5	0.003	0.190	0.026	0.031	0.006	0.204	#13	0.073	Not Required	Pass
6	0.011	0.686	0.140	0.072	0.030	0.808	#13	0.044	Not Required	Pass
7	0.012	0.425	0.222	0.068	0.043	0.473	#13	0.073	Not Required	Pass
8	0.007	0.143	0.138	0.040	0.011	0.169	#13	0.088	Not Required	Pass
9	0.006	0.076	0.110	0.006	0.007	0.160	#13	0.198	Not Required	Pass
10	0.013	0.645	0.222	0.065	0.040	0.664	#13	0.078	Not Required	Pass
11	0.006	0.126	0.141	0.042	0.011	0.149	#21	0.088	Not Required	Pass
12	0.003	0.422	0.286	0.086	0.056	0.709	#13	0.079	Not Required	Pass
13	0.008	0.082	0.296	0.053	0.014	0.333	#21	0.265	Not Required	Pass
14	0.007	0.062	0.292	0.050	0.014	0.304	#21	0.177	Not Required	Pass
15	0.000	0.004	0.005	0.006	0.001	0.008	#21	Not Required	Not Required	Pass
16	0.000	0.004	0.005	0.006	0.001	0.008	#21	Not Required	Not Required	Pass
101	0.043	0.749	0.007	0.046	0.001	0.774	#13	0.503	Not Required	Pass
102	0.004	0.402	0.331	0.090	0.061	0.735	#13	0.034	Not Required	Pass
103	0.011	0.751	0.068	0.075	0.004	0.809	#13	0.044	Not Required	Pass
104	0.010	0.783	0.212	0.079	0.035	0.888	#13	0.078	Not Required	Pass
105	0.011	0.466	0.232	0.075	0.047	0.515	#13	0.073	Not Required	Pass
106	0.010	0.738	0.068	0.074	0.006	0.781	#13	0.044	Not Required	Pass
107	0.010	0.460	0.222	0.074	0.045	0.504	#13	0.073	Not Required	Pass
108	0.007	0.088	0.131	0.047	0.011	0.193	#21	0.088	Not Required	Pass
109	0.016	0.051	0.075	0.001	0.000	0.131	#13	0.198	Not Required	Pass
110	0.010	0.742	0.216	0.075	0.036	0.855	#13	0.078	Not Required	Pass
111	0.006	0.068	0.135	0.046	0.011	0.187	#21	0.088	Not Required	Pass
112	0.004	0.375	0.322	0.085	0.061	0.698	#13	0.053	Not Required	Pass
113	0.008	0.244	0.302	0.061	0.014	0.463	#21	0.265	Not Required	Pass
114	0.009	0.286	0.299	0.064	0.014	0.512	#13	0.265	Not Required	Pass
115	0.033	0.445	0.160	0.050	0.011	0.574	#13	0.858	Not Required	Pass
116	0.020	0.429	0.162	0.053	0.011	0.559	#13	0.858	Not Required	Pass
201	0.042	0.724	0.000	0.044	0.000	0.745	#13	0.503	Not Required	Pass
202	0.003	0.374	0.311	0.084	0.058	0.686	#13	0.034	Not Required	Pass
203	0.010	0.730	0.065	0.073	0.004	0.782	#13	0.044	Not Required	Pass
204	0.010	0.712	0.204	0.071	0.034	0.813	#13	0.078	Not Required	Pass
205	0.010	0.453	0.210	0.073	0.044	0.406	#13	0.073	Not Required	Pass

205	0.010	0.453	0.219	0.073	0.044	0.490	#13	0.073	Not Required	Pass
206	0.010	0.729	0.065	0.073	0.004	0.782	#13	0.044	Not Required	Pass
207	0.010	0.453	0.219	0.073	0.044	0.495	#13	0.073	Not Required	Pass
208	0.007	0.069	0.127	0.045	0.011	0.175	#21	0.088	Not Required	Pass
209	0.015	0.049	0.070	0.001	0.000	0.124	#13	0.198	Not Required	Pass
210	0.010	0.712	0.204	0.071	0.034	0.813	#13	0.078	Not Required	Pass
211	0.006	0.075	0.132	0.047	0.011	0.186	#21	0.088	Not Required	Pass
212	0.003	0.374	0.311	0.084	0.058	0.686	#13	0.034	Not Required	Pass
213	0.008	0.259	0.285	0.058	0.014	0.460	#21	0.265	Not Required	Pass
214	0.009	0.253	0.279	0.056	0.014	0.444	#13	0.265	Not Required	Pass
215	0.031	0.301	0.162	0.047	0.011	0.431	#13	0.858	Not Required	Pass
216	0.020	0.289	0.162	0.045	0.011	0.417	#13	0.858	Not Required	Pass
301	0.043	0.749	0.007	0.046	0.001	0.774	#13	0.503	Not Required	Pass
302	0.004	0.375	0.321	0.085	0.061	0.698	#13	0.053	Not Required	Pass
303	0.010	0.738	0.068	0.074	0.006	0.781	#13	0.044	Not Required	Pass
304	0.010	0.743	0.216	0.075	0.036	0.855	#13	0.078	Not Required	Pass
305	0.010	0.460	0.222	0.074	0.045	0.504	#13	0.073	Not Required	Pass
306	0.011	0.751	0.068	0.075	0.004	0.809	#13	0.044	Not Required	Pass
307	0.011	0.466	0.232	0.075	0.047	0.515	#13	0.073	Not Required	Pass
308	0.007	0.078	0.144	0.053	0.011	0.201	#13	0.088	Not Required	Pass
309	0.016	0.051	0.075	0.001	0.000	0.130	#13	0.198	Not Required	Pass
310	0.010	0.783	0.212	0.079	0.035	0.888	#13	0.078	Not Required	Pass
311	0.006	0.071	0.146	0.050	0.011	0.185	#21	0.088	Not Required	Pass
312	0.004	0.402	0.331	0.090	0.061	0.735	#13	0.034	Not Required	Pass
313	0.008	0.243	0.302	0.061	0.014	0.463	#21	0.265	Not Required	Pass
314	0.009	0.286	0.299	0.064	0.014	0.512	#13	0.265	Not Required	Pass
315	0.031	0.304	0.162	0.046	0.011	0.434	#13	0.858	Not Required	Pass
316	0.020	0.285	0.162	0.047	0.011	0.413	#13	0.858	Not Required	Pass
401	0.029	0.490	0.089	0.030	0.008	0.529	#13	0.503	Not Required	Pass
402	0.003	0.421	0.286	0.086	0.056	0.708	#13	0.079	Not Required	Pass
403	0.011	0.685	0.140	0.072	0.030	0.807	#13	0.044	Not Required	Pass
404	0.013	0.645	0.222	0.065	0.040	0.663	#13	0.078	Not Required	Pass
405	0.012	0.425	0.222	0.068	0.043	0.473	#13	0.073	Not Required	Pass
406	0.005	0.306	0.035	0.028	0.011	0.337	#13	0.044	Not Required	Pass
407	0.003	0.191	0.026	0.031	0.006	0.204	#13	0.073	Not Required	Pass
408	0.000	0.004	0.005	0.006	0.001	0.008	#21	Not Required	Not Required	Pass
409	0.006	0.076	0.110	0.006	0.007	0.159	#13	0.198	Not Required	Pass
410	0.004	0.311	0.123	0.032	0.023	0.434	#13	0.117	Not Required	Pass
411	0.000	0.004	0.005	0.006	0.001	0.008	#21	Not Required	Not Required	Pass
412	0.004	0.080	0.112	0.030	0.028	0.193	#13	0.052	Not Required	Pass
413	0.008	0.082	0.296	0.053	0.014	0.333	#21	0.177	Not Required	Pass
414	0.007	0.062	0.292	0.050	0.014	0.303	#21	0.265	Not Required	Pass
415	0.033	0.467	0.160	0.042	0.011	0.596	#13	0.858	Not Required	Pass
416	0.020	0.466	0.162	0.041	0.011	0.596	#13	0.858	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 <sub>y</sub>	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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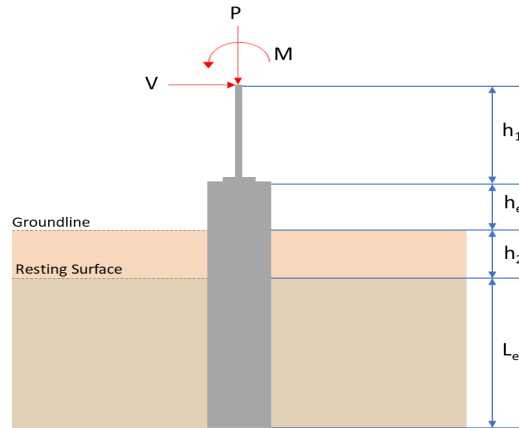
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 7.75$  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)	3.544	5.274
$V_x$ (kip)	-2.076	-3.450
$V_z$ (kip)	0.533	0.856
$M_x$ (kipft)	1.207	1.932
$M_z$ (kipft)	18.176	30.481

### 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,

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

$$H_o = \frac{(-2.076 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.692 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.176 \text{ kipft}) + ((-2.076 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.0587 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.8164 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.533 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.17767 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.207 \text{ kipft}) + ((0.533 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.40233 \text{ kipft/ft}$$

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:

$L_{e,z} = 5.1535 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(6.8164 \text{ ft}), (5.1535 \text{ ft})]$$

$$L_{e,req} = 6.816 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(6.816 \text{ ft})}{(7.75 \text{ ft})}$$

$$Ratio = 0.87948$$

Status: **PASS**  
Ratio: **0.880**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

$q$  - End-bearing pressure

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

$$q = \frac{(3.544 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.50137 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.50137 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25069$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.5833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.692 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 6.0587 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (6.0587 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (6.0587 \text{ kipft/ft})) + (4 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4063 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.0587 \text{ kipft/ft})) + (3 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (6.0587 \text{ kipft/ft})) + (2 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.17468 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.0587 \text{ kipft/ft})) + ((-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.0599 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4063 \text{ ft})}{2}$$

$$p_a = 0.40548 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17468 \text{ kip/ft}^2)}{(0.40548 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.4308$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0599 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91173$$

Status: **PASS**  
Ratio: **0.430**

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

**Considering z-direction:**

$H_o = 0.17767 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.40233 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.40233 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.17767 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.40233 \text{ kipft/ft})) + (4 \times (0.17767 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.6157 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.40233 \text{ kipft/ft})) + (3 \times (0.17767 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.40233 \text{ kipft/ft})) + (2 \times (0.17767 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.16315 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.40233 \text{ kipft/ft})) + ((0.17767 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 0.34233 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6157 \text{ ft})}{2}$$

$$p_a = 0.42118 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16315 \text{ kip/ft}^2)}{(0.42118 \text{ kip/ft}^2)}$$

$$\left( \frac{0.34233 \text{ kip/ft}^2}{1.1625 \text{ kip/ft}^2} \right)$$

$$\text{Ratio} = 0.38737$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

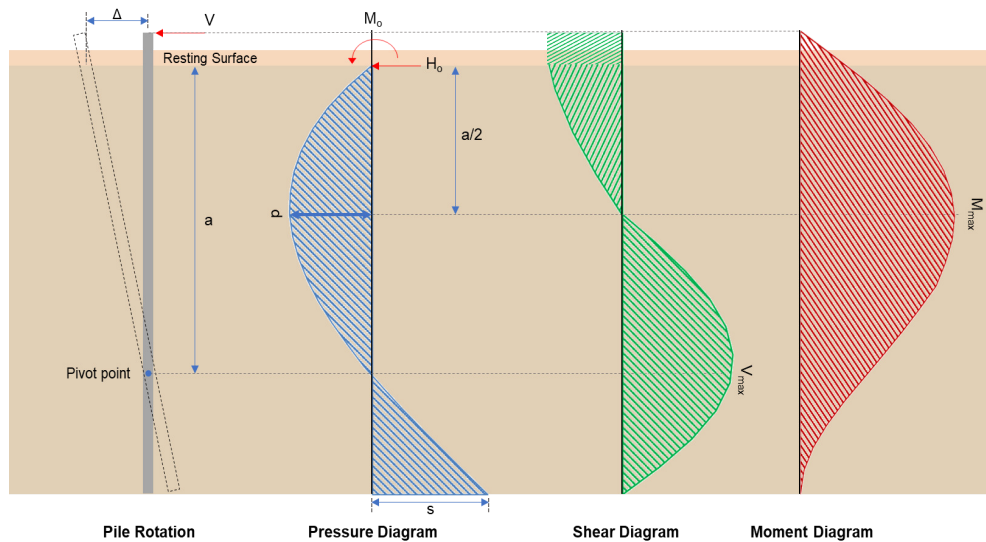
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.34233 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.29448$$

Status: **PASS**  
Ratio: **0.390**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.45 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.15 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(30.481 \text{ kipft}) + ((-3.45 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.16 \text{ kipft/ft}$$

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

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

$$E = \frac{(10.16 \text{ kipft/ft})}{(-1.15 \text{ kip/ft})}$$

$$E = 8.8351 \text{ ft}$$

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

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

$$a = \frac{(4 \times (10.16 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-1.15 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (10.16 \text{ kipft/ft})) + (4 \times (-1.15 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.405 \text{ ft}$$

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

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

$$V_{max} = ((-1.15 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.8351 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.405 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.8351 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.405 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2361 \text{ kip}$$

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

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

$$M_{max} = ((-1.15 \text{ kip/ft}) \times (36 \text{ in}) \times (7.75 \text{ ft})) \times \left[ \left( \frac{(8.8351 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.8351 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.8351 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.376 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.856 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.28533 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.932 \text{ kipft}) + ((0.856 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.644 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.644 \text{ kipft/ft})}{(0.28533 \text{ kip/ft})}$$

$$E = 2.257 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.644 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (0.28533 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.644 \text{ kipft/ft})) + (4 \times (0.28533 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.6161 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = \left( (0.28533 \text{ kip/ft}) \times (36 \text{ in}) \right) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.257 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.6161 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.257 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.6161 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 1.0162 \text{ kip}$$

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

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

$$M_{max} = \left( (0.28533 \text{ kip/ft}) \times (36 \text{ in}) \times (7.75 \text{ ft}) \right) \times \left[ \left( \frac{(2.257 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.6161 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.257 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.6161 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.257 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.6161 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.35 \text{ kipft}$$

### 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,

Table 22.4.2.1

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.274 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.208 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.208 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

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

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

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

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</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 style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]</math></p> <p><math>s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</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 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \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{(5.274 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.004206</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 36 \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 (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \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{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</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 (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.274 \text{ kip} \rightarrow 5274 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(5274 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.333 \text{ kip}$$

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)

$$V_{c,b} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.333 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.333 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.333 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.777 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 9.2361 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(9.2361 \text{ kip})}{(73.777 \text{ kip})}$$

$$Ratio = 0.12519$$

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

**Considering z-direction:**

$V_{max} = 1.0162 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(1.0162 \text{ kip})}{(73.777 \text{ kip})}$$

$$Ratio = 0.013774$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b  $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 33.376 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(33.376 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.5381$$

Status: **PASS**  
Ratio: **0.540**

**Considering z-direction:**

$M_{max} = 3.35 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

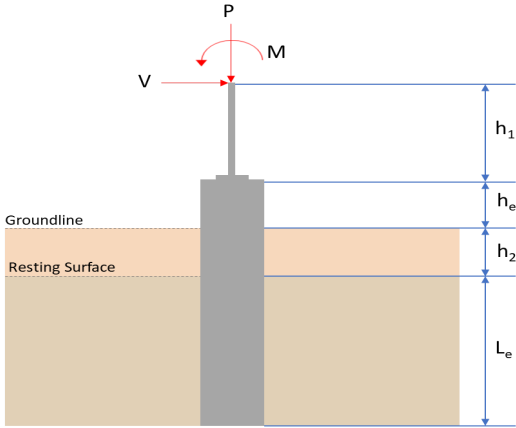
$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(3.35 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.054008$$

Status: **PASS**  
Ratio: **0.050**

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b></p> <p>Pile shape: round  <math>D = 36</math> in - Pile diameter  <math>L = 7.75</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resisting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="368 1061 1227 1162"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="655 1265 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>3.543</td> <td>5.273</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.076</td> <td>-3.450</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.532</td> <td>-0.853</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-1.209</td> <td>-1.934</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>18.174</td> <td>30.478</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	$P$ (kip)	3.543	5.273	$V_x$ (kip)	-2.076	-3.450	$V_z$ (kip)	-0.532	-0.853	$M_x$ (kipft)	-1.209	-1.934	$M_z$ (kipft)	18.174	30.478	
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	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.076 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.692 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p>																											

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

$$M_o = \frac{(18.174 \text{ kipft}) + ((-2.076 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.058 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.816 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.532 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.17733 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.209 \text{ kipft}) + ((-0.532 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.403 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.3009 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.816 \text{ ft}), (2.3009 \text{ ft})]$$

$$L_{e,req} = 6.816 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.816 \text{ ft})}{(7.75 \text{ ft})}$$

$$\text{Ratio} = 0.87948$$

Status: **PASS**  
Ratio: **0.880**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

$q$  - End-bearing pressure

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

$$q = \frac{(3.543 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.50123 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.50123 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25062$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.5833$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.692 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 6.058 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (6.058 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (6.058 \text{ kipft/ft})) + (4 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.4064 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.058 \text{ kipft/ft})) + (3 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (6.058 \text{ kipft/ft})) + (2 \times (-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.17461 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.058 \text{ kipft/ft})) + ((-0.692 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.0597 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.4064 \text{ ft})}{2}$$

$$p_a = 0.40548 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17461 \text{ kip/ft}^2)}{(0.40548 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.43063$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0597 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91155$$

Status: **PASS**  
Ratio: **0.430**

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

**Considering z-direction:**

$H_o = -0.17733 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.403 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.403 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.17733 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.403 \text{ kipft/ft})) + (4 \times (-0.17733 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.6152 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.403 \text{ kipft/ft})) + (3 \times (-0.17733 \text{ kip/ft}) \times (7.75 \text{ ft}))]^2}{(7.75 \text{ ft})^2 \times [(3 \times (0.403 \text{ kipft/ft})) + (2 \times (-0.17733 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = -0.080317 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.403 \text{ kipft/ft})) + ((-0.17733 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = -0.089183 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6152 \text{ ft})}{2}$$

$$p_a = 0.42114 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.080317 \text{ kip/ft}^2)}{(0.42114 \text{ kip/ft}^2)}$$

(continued)

$$Ratio = -0.19071$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

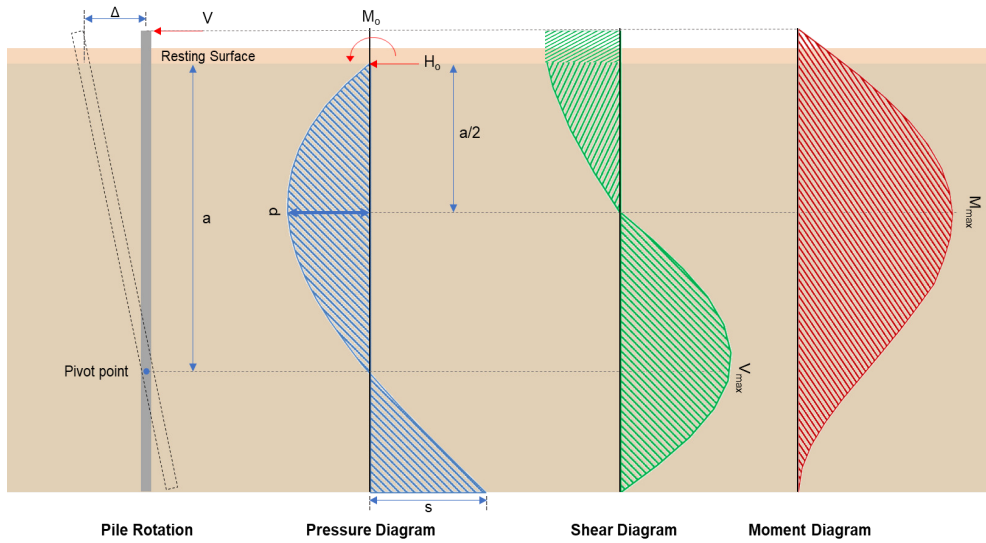
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.089183 \text{ kip/ft}^2)}{(1.1625 \text{ kip/ft}^2)}$$

$$Ratio = -0.076716$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.45 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.15 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(30.478 \text{ kipft}) + ((-3.45 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.159 \text{ kipft/ft}$$

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

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

$$E = \frac{(10.159 \text{ kipft/ft})}{(-1.15 \text{ kip/ft})}$$

$$E = 8.8342 \text{ ft}$$

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

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

$$a = \frac{(4 \times (10.159 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-1.15 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (10.159 \text{ kipft/ft})) + (4 \times (-1.15 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.405 \text{ ft}$$

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

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

$$V_{max} = ((-1.15 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.8342 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.405 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.8342 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.405 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.2354 \text{ kip}$$

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

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

$$M_{max} = ((-1.15 \text{ kip/ft}) \times (36 \text{ in}) \times (7.75 \text{ ft})) \times \left[ \left( \frac{(8.8342 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.8342 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.8342 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.405 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.374 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.853 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.28433 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.934 \text{ kipft}) + ((-0.853 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.64467 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.64467 \text{ kipft/ft})}{(-0.28433 \text{ kip/ft})}$$

$$E = 2.2673 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.64467 \text{ kipft/ft}) \times (7.75 \text{ ft})) + (3 \times (-0.28433 \text{ kip/ft}) \times (7.75 \text{ ft})^2)}{(6 \times (0.64467 \text{ kipft/ft})) + (4 \times (-0.28433 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.6155 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.28433 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.2673 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.6155 \text{ ft})}{(7.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.2673 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.6155 \text{ ft})}{(7.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 1.0146 \text{ kip}$$

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

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

$$M_{max} = ((-0.28433 \text{ kip/ft}) \times (36 \text{ in}) \times (7.75 \text{ ft})) \times \left[ \left( \frac{(2.2673 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.6155 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.2673 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left( \frac{(5.6155 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.2673 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left( \frac{(5.6155 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 3.3458 \text{ kipft}$$

### 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,

Table 22.4.2.1

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(5.273 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.209 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.209 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

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

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

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

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</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 style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]</math></p> <p><math>s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</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 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \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{(5.273 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0042052</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 36 \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 (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \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{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</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 (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 5.273 \text{ kip} \rightarrow 5273 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(5273 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.333 \text{ kip}$$

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)

$$V_{c,b} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.333 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.333 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.333 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.777 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 9.2354 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(9.2354 \text{ kip})}{(73.777 \text{ kip})}$$

$$Ratio = 0.12518$$

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

**Considering z-direction:**

$V_{max} = 1.0146 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(1.0146 \text{ kip})}{(73.777 \text{ kip})}$$

$$Ratio = 0.013752$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 33.374 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(33.374 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.53805$$

Status: **PASS**  
Ratio: **0.540**

**Considering z-direction:**

$M_{max} = 3.3458 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(3.3458 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.053941$$

Status: **PASS**  
Ratio: **0.050**

REFERENCES	CALCULATIONS	RESULTS
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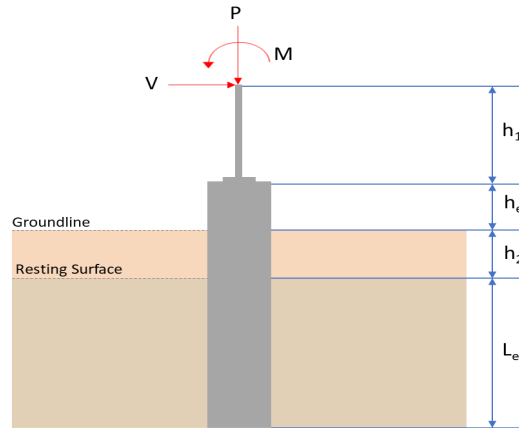
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 8.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	5.155	7.813
$V_x$ (kip)	-3.149	-5.236
$V_z$ (kip)	-0.046	-0.072
$M_x$ (kipft)	-0.123	-0.194
$M_z$ (kipft)	27.700	46.641

### 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,

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

$$H_o = \frac{(-3.149 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.0497 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(27.7 \text{ kipft}) + ((-3.149 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 9.2333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.4863 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.046 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.015333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((-0.046 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.041 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4511 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.4863 \text{ ft}), (1.4511 \text{ ft})]$$

$$L_{e,req} = 7.486 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.486 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.88071$$

Status: **PASS**  
Ratio: **0.880**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

$q$  - End-bearing pressure

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

$$q = \frac{(5.155 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.72928 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.72928 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.36464$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -1.0497 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 9.2333 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (9.2333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.2333 \text{ kipft/ft})) + (4 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9442 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (9.2333 \text{ kipft/ft})) + (3 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.2333 \text{ kipft/ft})) + (2 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.171 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (9.2333 \text{ kipft/ft})) + ((-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2451 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9442 \text{ ft})}{2}$$

$$p_a = 0.44581 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.171 \text{ kip/ft}^2)}{(0.44581 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.38356$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2451 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97653$$

Status: **PASS**  
Ratio: **0.380**

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

**Considering z-direction:**

$H_o = -0.015333 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.041 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.041 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.015333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.041 \text{ kipft/ft})) + (4 \times (-0.015333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1479 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.041 \text{ kipft/ft})) + (3 \times (-0.015333 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.041 \text{ kipft/ft})) + (2 \times (-0.015333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.0061028 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.041 \text{ kipft/ft})) + ((-0.015333 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.0063051 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1479 \text{ ft})}{2}$$

$$p_a = 0.46109 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0061028 \text{ kip/ft}^2)}{(0.46109 \text{ kip/ft}^2)}$$

$$\left( \frac{0.00159 \text{ kip/ft}}{1.275 \text{ kip/ft}^2} \right)$$

$$\text{Ratio} = -0.013236$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

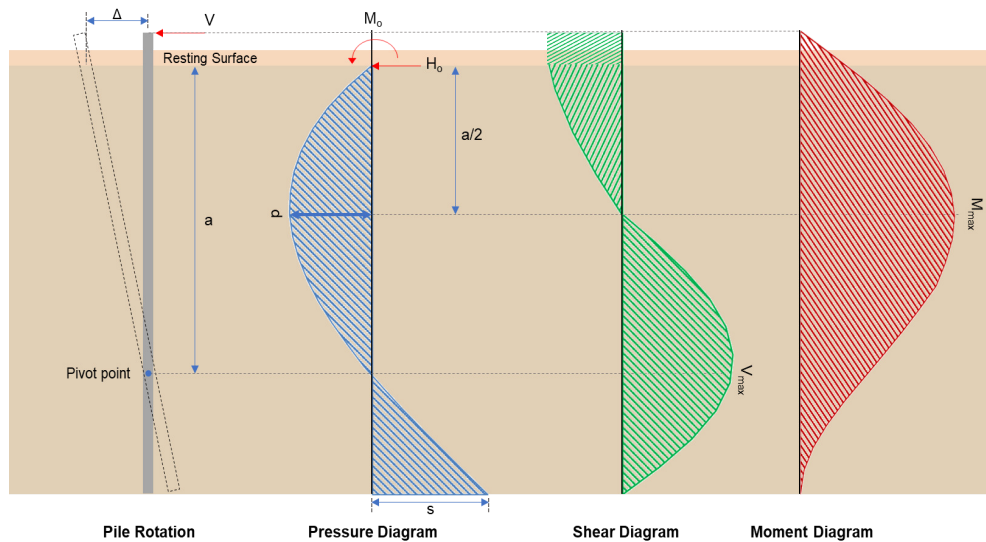
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0063051 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0049452$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-5.236 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.7453 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(46.641 \text{ kipft}) + ((-5.236 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 15.547 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.547 \text{ kipft/ft})}{(-1.7453 \text{ kip/ft})}$$

$$E = 8.9078 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.547 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.7453 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (15.547 \text{ kipft/ft})) + (4 \times (-1.7453 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9421 \text{ ft}$$

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

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

$$V_{max} = ((-1.7453 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.9421 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.9421 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.167 \text{ kip}$$

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

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

$$M_{max} = ((-1.7453 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(8.9078 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.946 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.072 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.024 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.194 \text{ kipft}) + ((-0.072 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.064667 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.064667 \text{ kipft/ft})}{(-0.024 \text{ kip/ft})}$$

$$E = 2.6944 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.064667 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.024 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.064667 \text{ kipft/ft})) + (4 \times (-0.024 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1467 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.024 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.6944 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.1467 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.6944 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.1467 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.088696 \text{ kip}$$

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

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

$$M_{max} = ((-0.024 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(2.6944 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.1467 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.6944 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.1467 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.6944 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.1467 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.32268 \text{ kipft}$$

### 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,

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.813 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.129 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.129 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

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

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

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

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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 = \frac{\quad}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</math></p> <p><math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> <p style="text-align: center;"><math>s_{rebar} = Max [1.5, (1.5 d_{bar})]</math></p> <p style="text-align: center;"><math>s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum center-to-center spacing of ties,</p> <p style="text-align: center;"><math>s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), D]</math></p> <p style="text-align: center;"><math>s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</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 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \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{(7.813 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0062309</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 = 36 \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 (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \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{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</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 (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.813 \text{ kip} \rightarrow 7813 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7813 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.764 \text{ kip}$$

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)

$$V_{c,b} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.764 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.764 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.764 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.058 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 13.167 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(13.167 \text{ kip})}{(74.058 \text{ kip})}$$

$$Ratio = 0.17779$$

Status: **PASS**  
Ratio: **0.180**

**Considering z-direction:**

$V_{max} = 0.088696 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.088696 \text{ kip})}{(74.058 \text{ kip})}$$

$$Ratio = 0.0011977$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b  $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 51.946 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.946 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.83747$$

Status: **PASS**  
Ratio: **0.840**

**Considering z-direction:**

$M_{max} = 0.32268 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

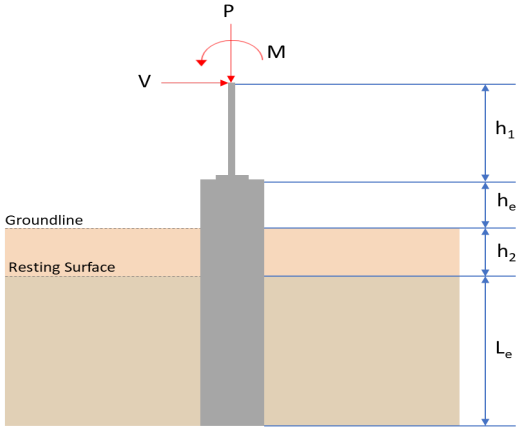
$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.32268 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0052023$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b></p> <p>Pile shape: round  <math>D = 36</math> in - Pile diameter  <math>L = 8.5</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resisting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="368 1061 1225 1162"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.989</td> <td>7.528</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.997</td> <td>-4.994</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.000</td> <td>-0.001</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.003</td> <td>-0.006</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>26.693</td> <td>45.045</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	$P$ (kip)	4.989	7.528	$V_x$ (kip)	-2.997	-4.994	$V_z$ (kip)	0.000	-0.001	$M_x$ (kipft)	-0.003	-0.006	$M_z$ (kipft)	26.693	45.045	
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$M_z$ (kipft)	26.693	45.045																										
	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.997 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.999 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p>																											

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

$$M_o = \frac{(26.693 \text{ kipft}) + ((-2.997 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.8977 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.4617 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.003 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.001 \text{ kipft/ft}$$

$L_e$  - Required depth of embedment in earth,

$$L_e = 2.66 \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.66 \times \sqrt[3]{\frac{(0.001 \text{ kipft/ft})}{(150 \text{ psf/ft})}}$$

$$L_e = 0.50063 \text{ ft}$$

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.4617 \text{ ft}), (0.50063 \text{ ft})]$$

$$L_{e,req} = 7.462 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.462 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.87788$$

Status: **PASS**  
Ratio: **0.880**

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = \pi \cdot r^2 = \left( \frac{D}{2} \right)^2 \cdot \pi$$

$$A = 7.0686 \text{ ft}^2$$

$q$  - End-bearing pressure

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

$$q = \frac{(4.989 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.7058 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.7058 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.3529$$

Status: **PASS**  
Ratio: **0.350**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.999 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 8.8977 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (8.8977 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.999 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (8.8977 \text{ kipft/ft})) + (4 \times (-0.999 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9421 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (8.8977 \text{ kipft/ft})) + (3 \times (-0.999 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (8.8977 \text{ kipft/ft})) + (2 \times (-0.999 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.17184 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (8.8977 \text{ kipft/ft})) + ((-0.999 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2137 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times (5.9421 \text{ ft})$$

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

$$p_a = 0.44566 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17184 \text{ kip/ft}^2)}{(0.44566 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.38558$$

Status: **PASS**  
Ratio: **0.390**

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2137 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95191$$

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

**Considering z-direction:**

$H_o = 0 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.001 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.001 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.001 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.6667 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.001 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.001 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.000086957 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.001 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.0002609 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6667 \text{ ft})}{2}$$

$$p_a = 0.425 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$P_a$

$$Ratio = \frac{(0.000086957 \text{ kip/ft}^2)}{(0.425 \text{ kip/ft}^2)}$$

$$Ratio = 0.00020461$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

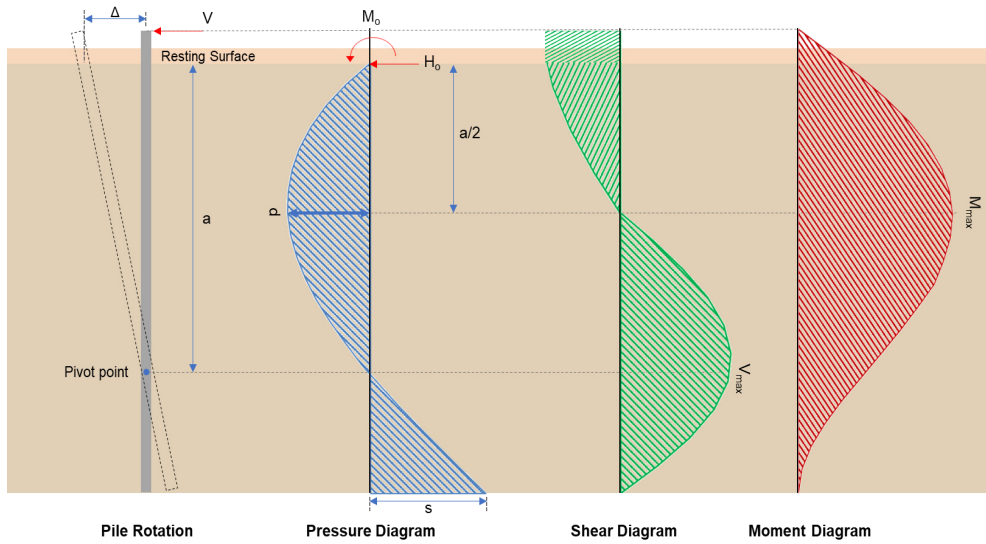
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0002609 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.00020463$$

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,

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

$$H_o = \frac{(-4.994 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.6647 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(45.045 \text{ kipft}) + ((-4.994 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 15.015 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.015 \text{ kipft/ft})}{(-1.6647 \text{ kip/ft})}$$

$$E = 9.0198 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.015 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.6647 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (15.015 \text{ kipft/ft})) + (4 \times (-1.6647 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.94 \text{ ft}$$

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

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

$$V_{max} = ((-1.6647 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (9.0198 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.94 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (9.0198 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.94 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.674 \text{ kip}$$

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

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

$$M_{max} = ((-1.6647 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(9.0198 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.94 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (9.0198 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.94 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (9.0198 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.94 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 50.038 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.001 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.00033333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.006 \text{ kipft}) + ((-0.001 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.002 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.002 \text{ kipft/ft})}{(-0.00033333 \text{ kip/ft})}$$

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

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

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

$$a = \frac{(4 \times (0.002 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.00033333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.002 \text{ kipft/ft})) + (4 \times (-0.00033333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0107 \text{ ft}$$

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

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

$$V_{max} = ((-0.00033333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (6 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0107 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (6 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0107 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0019121 \text{ kip}$$

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

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

$$M_{max} = ((-0.00033333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(6 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0107 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (6 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0107 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (6 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0107 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.0073644 \text{ kipft}$$

### 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,

Table 22.4.2.1

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.528 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.138 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.138 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

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

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

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

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

	$Ratio = \frac{A_{min}}{A_{st}}$ $Ratio = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $Ratio = 0.99533$ <p>25.2.3 <math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> $s_{rebar} = Max [1.5, (1.5 d_{bar})]$ $s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>25.7.2.2 Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p>25.7.2.1 <math>s_{ties}</math> - Maximum center-to-center spacing of ties,</p> $s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</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> $\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$ $\phi P_N = 1253.9 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.528 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0060036$	<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 = 36 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \cdot \sqrt{f'_c} \cdot b_w \cdot d$	

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.528 \text{ kip} \rightarrow 7528 \text{ lbf}$ ,  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7528 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.716 \text{ kip}$$

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)

$$V_{c,b} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (75.716 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.716 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN} [(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.716 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.026 \text{ kip}$$

**Considering x-direction:**

$V_x = 19.674 \text{ kip}$  - Maximum shear force in the x-direction

$V_{max} = 12.674 \text{ kip}$  - Maximum shear force in the x-direction,  
 Ratio - Capacity

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

$$Ratio = \frac{(12.674 \text{ kip})}{(74.026 \text{ kip})}$$

$$Ratio = 0.17121$$

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

**Considering z-direction:**

$V_{max} = 0.0019121 \text{ kip}$  - Maximum shear force in the z-direction,  
 Ratio - Capacity

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

$$Ratio = \frac{(0.0019121 \text{ kip})}{(74.026 \text{ kip})}$$

$$Ratio = 0.00002583$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b  $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 50.038 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(50.038 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.80672$$

Status: **PASS**  
 Ratio: **0.810**

**Considering z-direction:**

$M_{max} = 0.0073644$  kipft - Maximum moment in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.0073644 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.00011873$$

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

REFERENCES	CALCULATIONS	RESULTS
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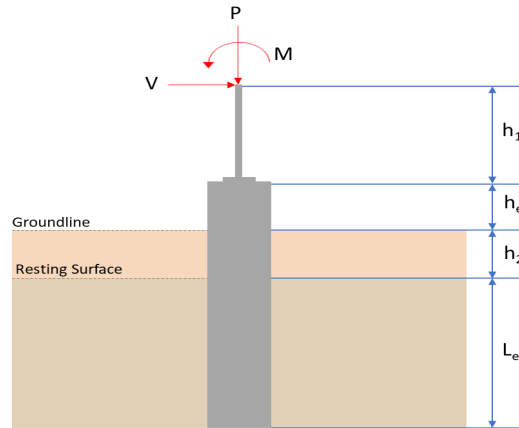
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 8.5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	5.155	7.813
$V_x$ (kip)	-3.149	-5.236
$V_z$ (kip)	0.045	0.070
$M_x$ (kipft)	0.115	0.181
$M_z$ (kipft)	27.700	46.641

### 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,

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

$$H_o = \frac{(-3.149 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.0497 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(27.7 \text{ kipft}) + ((-3.149 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 9.2333 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.4863 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.045 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.015 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.115 \text{ kipft}) + ((0.045 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.038333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9658 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.4863 \text{ ft}), (1.9658 \text{ ft})]$$

$$L_{e,req} = 7.486 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.486 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.88071$$

Status: **PASS**  
Ratio: **0.880**

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

$q$  - End-bearing pressure

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

$$q = \frac{(5.155 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.72928 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.72928 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.36464$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -1.0497 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 9.2333 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (9.2333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (9.2333 \text{ kipft/ft})) + (4 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9442 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (9.2333 \text{ kipft/ft})) + (3 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (9.2333 \text{ kipft/ft})) + (2 \times (-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.171 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (9.2333 \text{ kipft/ft})) + ((-1.0497 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2451 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.9442 \text{ ft})}{2}$$

$$p_a = 0.44581 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.171 \text{ kip/ft}^2)}{(0.44581 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.38356$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2451 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97653$$

Status: **PASS**  
Ratio: **0.380**

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

**Considering z-direction:**

$H_o = 0.015 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.038333 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.038333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.015 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.038333 \text{ kipft/ft})) + (4 \times (0.015 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1548 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.038333 \text{ kipft/ft})) + (3 \times (0.015 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.038333 \text{ kipft/ft})) + (2 \times (0.015 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.012652 \text{ kip/ft}^2$$

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

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.038333 \text{ kipft/ft})) + ((0.015 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.026634 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1548 \text{ ft})}{2}$$

$$p_a = 0.46161 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.012652 \text{ kip/ft}^2)}{(0.46161 \text{ kip/ft}^2)}$$

(continued)

$$Ratio = 0.027409$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

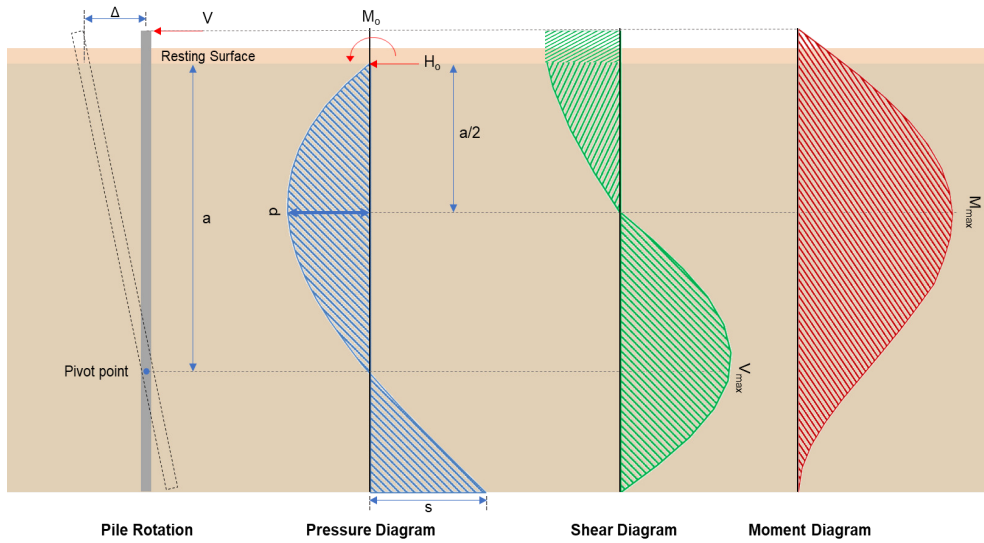
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.026634 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.020889$$

Status: **PASS**  
Ratio: **0.030**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-5.236 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.7453 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(46.641 \text{ kipft}) + ((-5.236 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 15.547 \text{ kipft/ft}$$

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

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

$$E = \frac{(15.547 \text{ kipft/ft})}{(-1.7453 \text{ kip/ft})}$$

$$E = 8.9078 \text{ ft}$$

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

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

$$a = \frac{(4 \times (15.547 \text{ kip/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.7453 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (15.547 \text{ kip/ft})) + (4 \times (-1.7453 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.9421 \text{ ft}$$

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

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

$$V_{max} = ((-1.7453 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.9421 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.9421 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 13.167 \text{ kip}$$

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

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

$$M_{max} = ((-1.7453 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(8.9078 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.9078 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.9421 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 51.946 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.07 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.023333 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.181 \text{ kipft}) + ((0.07 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.060333 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.060333 \text{ kipft/ft})}{(0.023333 \text{ kip/ft})}$$

$$E = 2.5857 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.060333 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.023333 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.060333 \text{ kipft/ft})) + (4 \times (0.023333 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1531 \text{ ft}$$

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

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

$$\left[ \frac{L_e}{L_e} \right] / \left[ \frac{L_e}{L_e} \right]$$

$$V_{max} = ((0.023333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.5857 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.1531 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.5857 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.1531 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.084677 \text{ kip}$$

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

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

$$M_{max} = ((0.023333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(2.5857 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.1531 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.5857 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.1531 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.5857 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.1531 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.30713 \text{ kipft}$$

### 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,

$\alpha = 0.85$  - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{(7.813 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2)), \frac{(7.813 \text{ kip})}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.129 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.129 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

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

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

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

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{1.8322 \text{ in}^2}{1.8408 \text{ in}^2}$$

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 = \frac{\lambda}{(1.8408 \text{ in}^2)}</math></p> <p style="text-align: center;"><math>Ratio = 0.99533</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> <p style="text-align: center;"><math>s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]</math></p> <p style="text-align: center;"><math>s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10<math>\emptyset</math>: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum center-to-center spacing of ties,</p> <p style="text-align: center;"><math>s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), D]</math></p> <p style="text-align: center;"><math>s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]</math></p> <p style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>6 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>1.000</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 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 1253.9 \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{(7.813 \text{ kip})}{(1253.9 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0062309</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 = 36 \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 (36 \text{ in})</math></p> <p style="text-align: center;"><math>d = 28.8 \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{(28.8 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.71796</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 (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})</math></p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 7.813 \text{ kip} \rightarrow 7813 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7813 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.764 \text{ kip}$$

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)

$$V_{c,b} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$$

$$V_{c,b} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.764 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.764 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

$A_v$  - 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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.764 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.058 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 13.167 \text{ kip}$  - Maximum shear force in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(13.167 \text{ kip})}{(74.058 \text{ kip})}$$

$$Ratio = 0.17779$$

Status: **PASS**  
Ratio: **0.180**

**Considering z-direction:**

$V_{max} = 0.084677 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.084677 \text{ kip})}{(74.058 \text{ kip})}$$

$$Ratio = 0.0011434$$

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

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

$S_m$  - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b  $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 51.946 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(51.946 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.83747$$

Status: **PASS**  
Ratio: **0.840**

**Considering z-direction:**

$M_{max} = 0.30713 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{\quad}{\phi M_n}$$

$$Ratio = \frac{(0.30713 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0049516$$

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