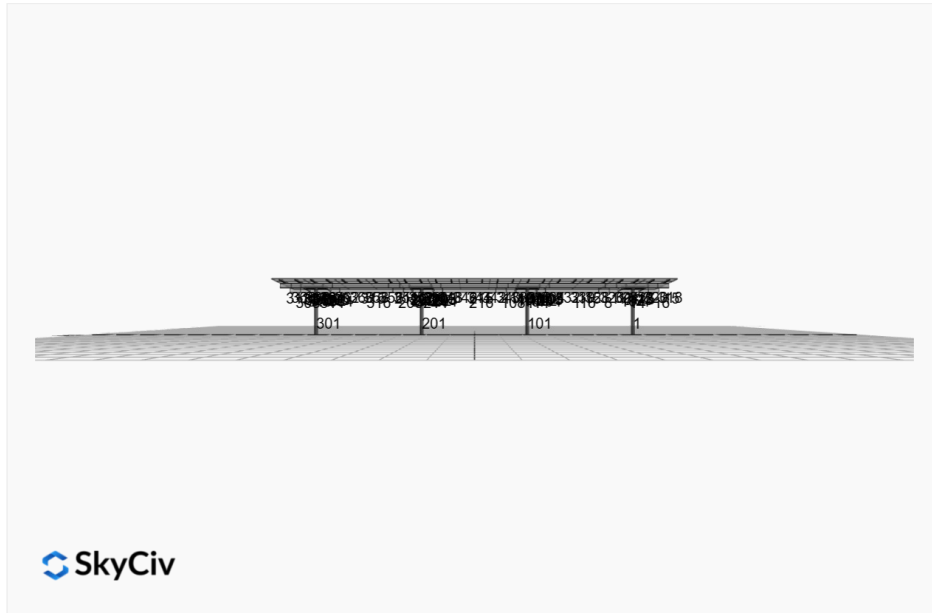


# Project Details



**Project Name:** MTSOLAR\_CKEG7EJIAAD5-Overlook at  
uplands-5 x 13 - 4pole  
**Location:** 8660 Clay St, Westminster, CO 80031, USA  
**Unique ID:** 4P-19.75-6TOP-XD-24-L-5Hx13W-886K  
**Dealer:** \_\_\_\_\_

**Date:** Thu May 01 2025  
**Number of Modules:** 65  
**Number of Poles:** 4  
**Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	16.88 ft
<b>Array Dimensions E/W</b>	71.50 ft
<b>Winter Tilt Angle</b>	5
<b>Front Edge Clearance</b>	8 ft

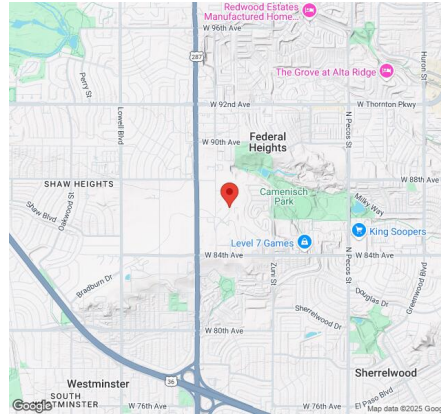
## MT Solar Bill of Materials (4P-19.75-6TOP-XD-24-L-5Hx13W-886K)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-24	24IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

## Rail Bill of Materials

Part	Qty
Rails (203in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

## Site Details:



**Site Address:** 8660 Clay St, Westminster, CO 80031, USA

### Array Specification

<b>Duty Classification:</b>	XD
<b>Module Width:</b>	40.00 in
<b>Module Length:</b>	65.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	13
<b>Total Number of Modules:</b>	65
<b>Winter Tilt Angle:</b>	5
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	9.47 ft
<b>Total Frame Length:</b>	70.75 ft
<b>Module Info/Notes:</b>	
<b>Array Dimensions N/S:</b>	16.88 ft
<b>Array Dimensions E/W:</b>	71.50 ft
<b>Rail Length:</b>	202.50 in
<b>Rail Spacing:</b>	2.75 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	8.74 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	19.75 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 4.50 ft Pile 2: 4.75 ft Pile 3: 4.75 ft Pile 4: 4.50 ft
<b>Foundation Volume:</b>	10.963 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	8660 Clay St, Westminster, CO 80031, USA
<b>Wind Speed:</b>	130 mph

**Snow Load:**

40 psf

### **Design Disclaimer**

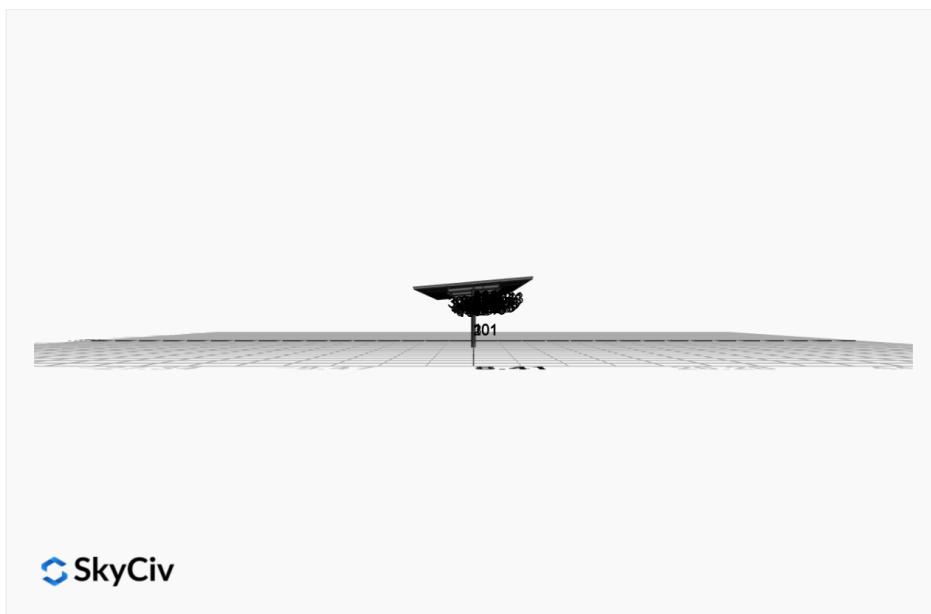
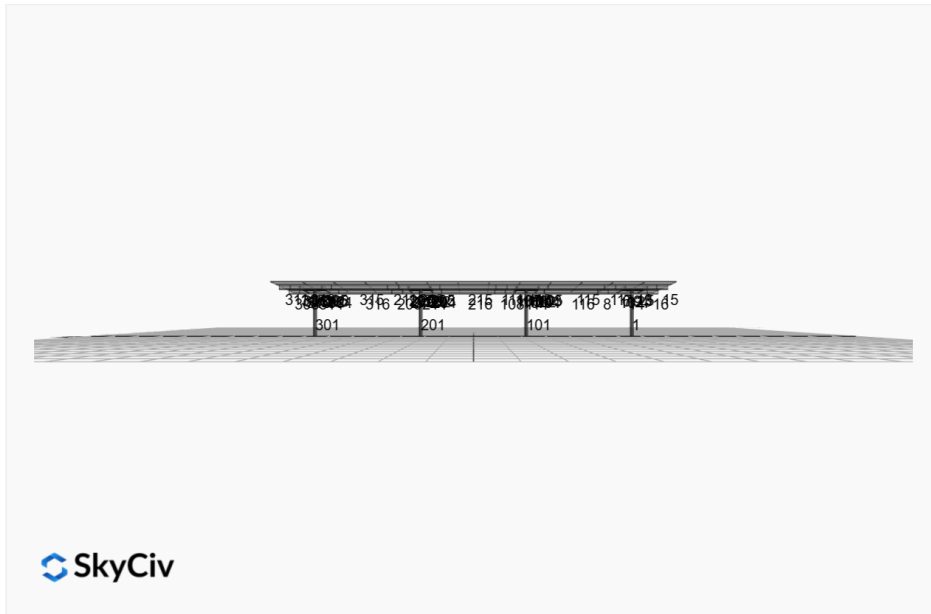
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

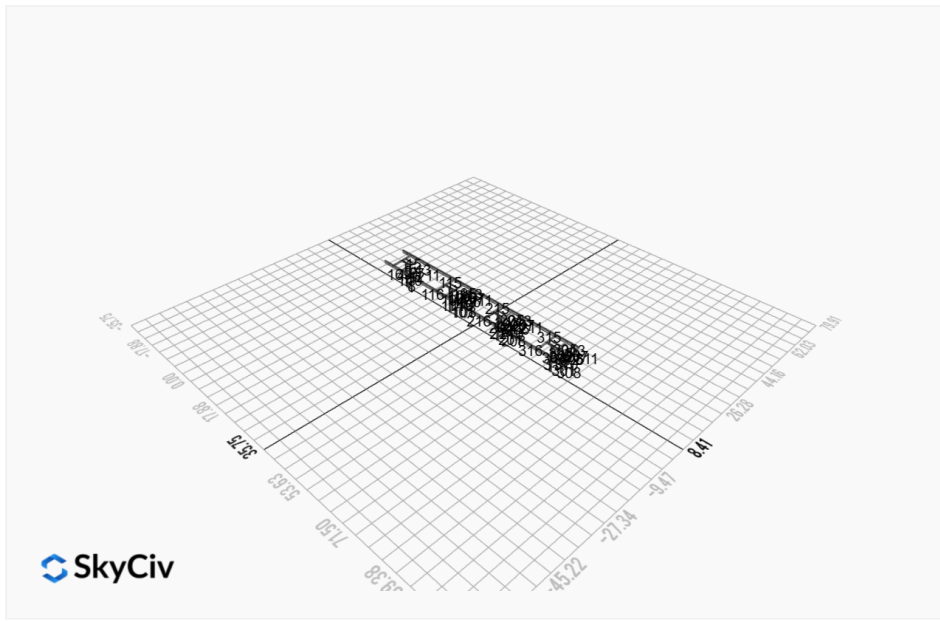
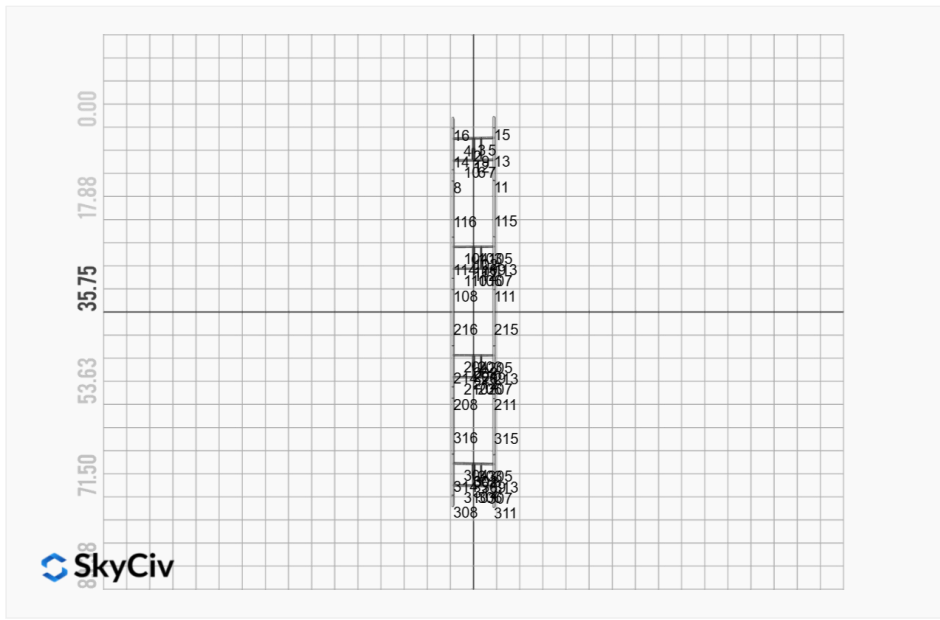
## AutoDesigner Input

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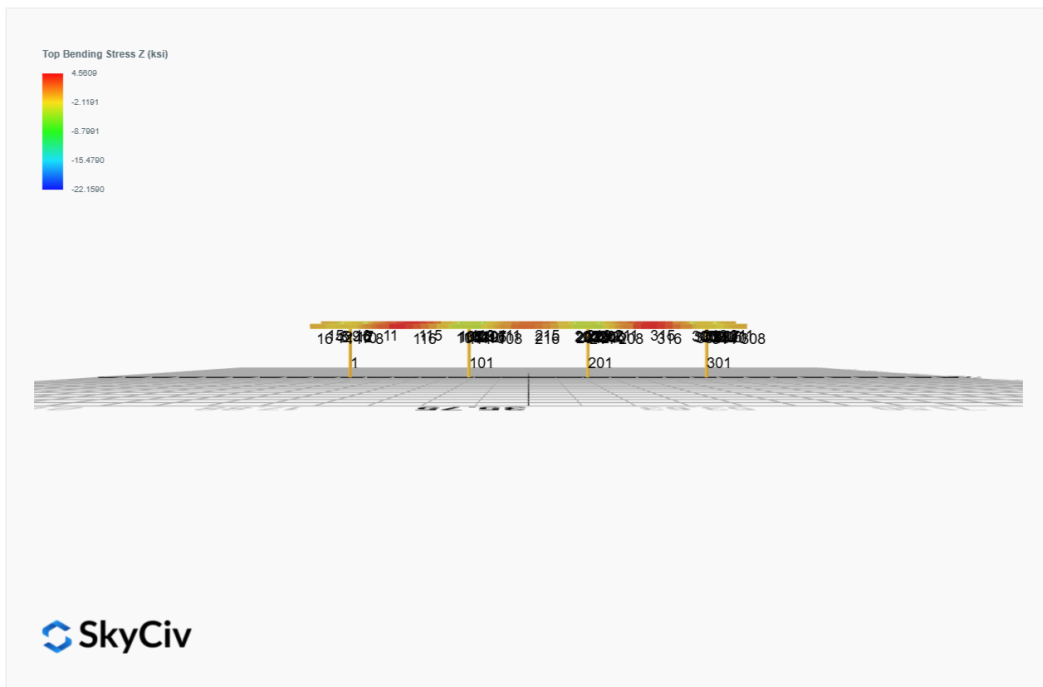
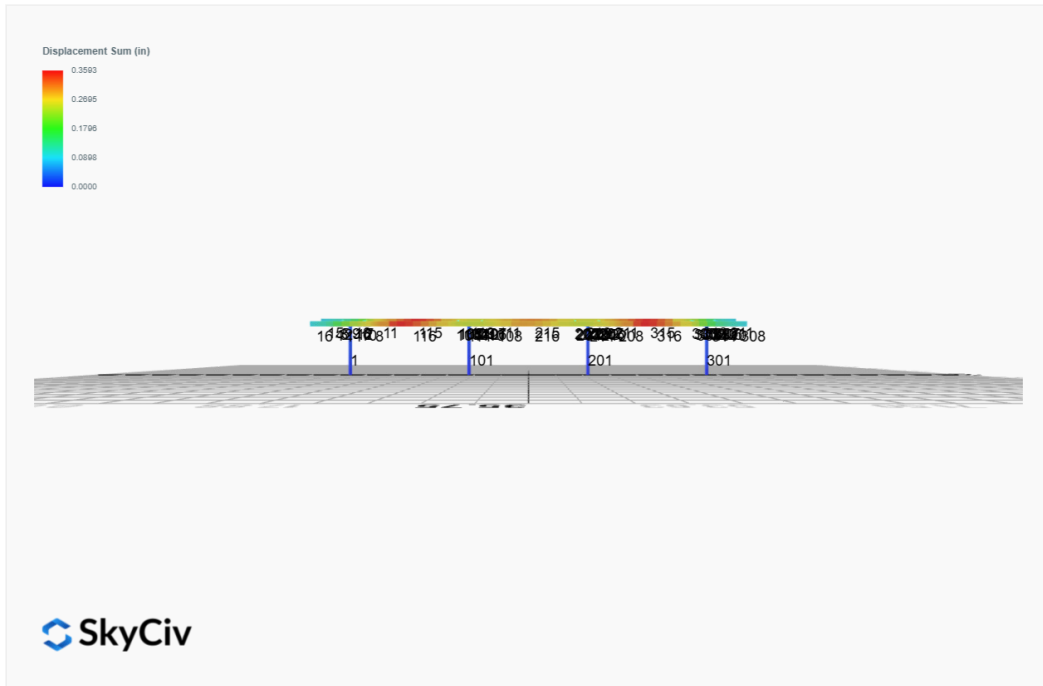
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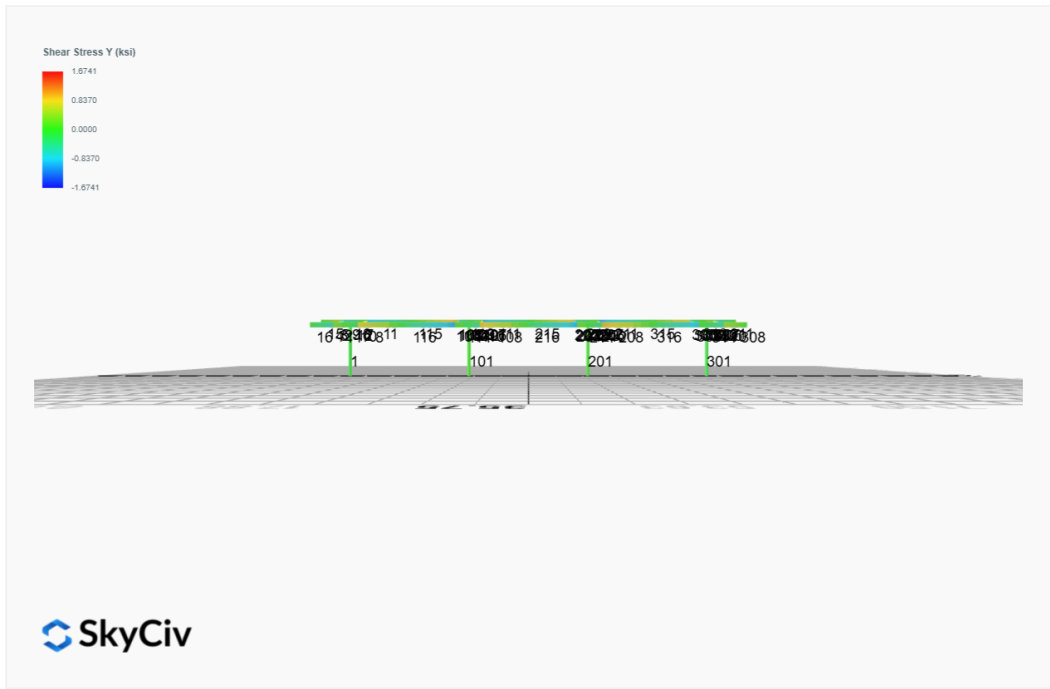
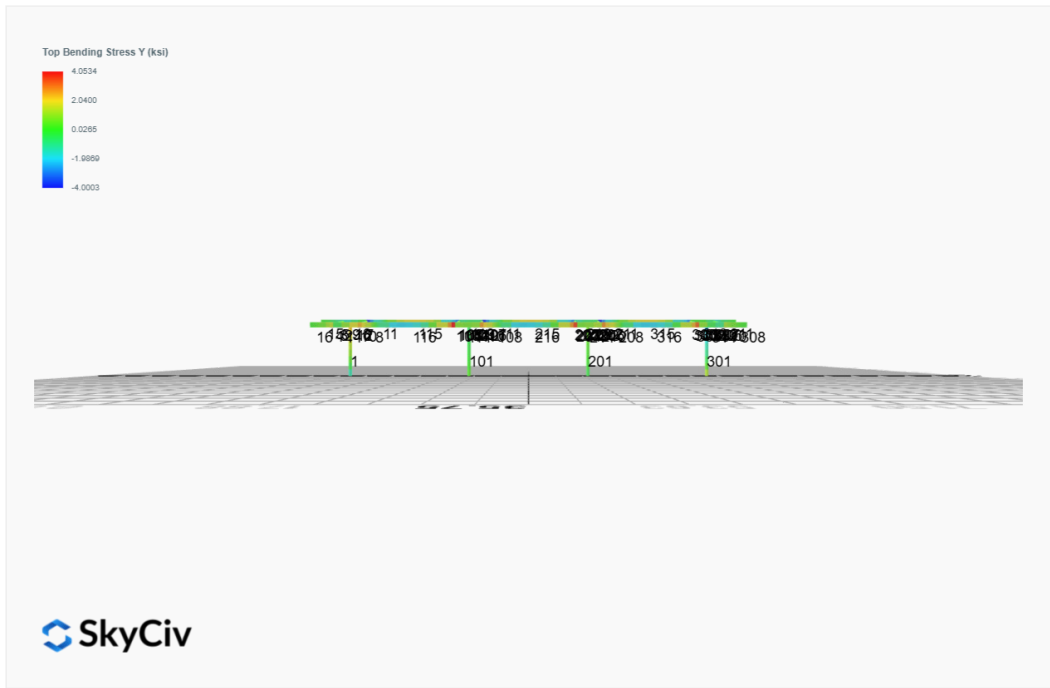
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

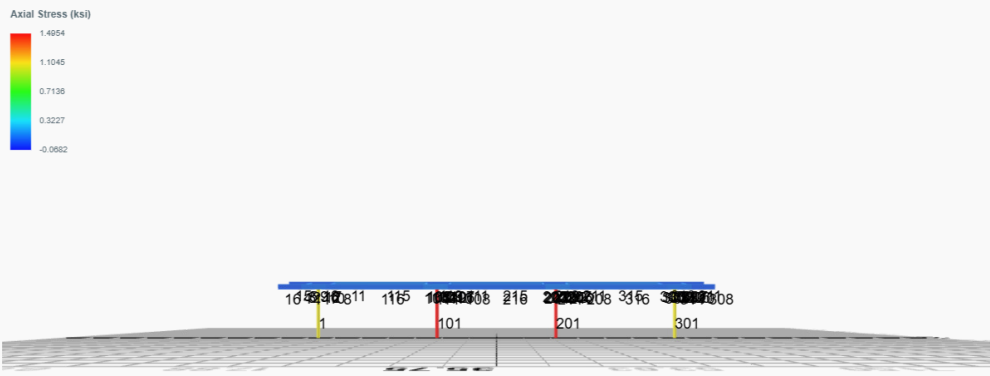




# FEM Results (Envelope Worst Case for each member)







## 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	0.0060	1.9716	0.0741	0.1942	-0.0067	-0.0135
ULS: 2. D + L	0.0060	1.9716	0.0741	0.1942	-0.0067	-0.0135
ULS: 3. D + (S or Lr or R)	0.0298	8.0119	0.3670	0.9635	-0.0332	-0.1894
ULS: 3. D + (S or Lr or R)	0.0060	1.9716	0.0741	0.1942	-0.0067	-0.0135
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0239	6.5018	0.2938	0.7712	-0.0265	-0.1454
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0060	1.9716	0.0741	0.1942	-0.0067	-0.0135
ULS: 5b. D + 0.7E	0.0060	1.9716	0.0741	0.1942	-0.0067	-0.0135
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0239	6.5018	0.2938	0.7712	-0.0265	-0.1454
ULS: 8. 0.6D + 0.7E	0.0036	1.1829	0.0444	0.1165	-0.0040	-0.0081
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2054	4.3107	0.1891	0.4941	-0.0350	2.3639
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2054	4.3107	0.1891	0.4941	-0.0350	2.3639
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0542	1.3423	0.0464	0.1218	-0.0035	2.0592
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1576	0.4828	-0.0051	-0.0111	0.0199	-6.8808
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1347	8.2561	0.3800	0.9962	-0.0478	1.6376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1347	8.2561	0.3800	0.9962	-0.0478	1.6376
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0599	6.0298	0.2730	0.7169	-0.0241	1.4091
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1375	5.3852	0.2344	0.6173	-0.0066	-5.2959
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1525	3.7259	0.1603	0.4191	-0.0280	1.7696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1525	3.7259	0.1603	0.4191	-0.0280	1.7696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0421	1.4996	0.0533	0.1399	-0.0043	1.5410
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1197	0.8550	0.0147	0.0402	0.0133	-5.1640
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2078	3.5220	0.1594	0.4165	-0.0324	2.3693
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2078	3.5220	0.1594	0.4165	-0.0324	2.3693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0517	0.5536	0.0168	0.0441	-0.0008	2.0646
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1552	-0.3058	-0.0347	-0.0887	0.0226	-6.8754

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.9791
Shear X	-0.3523
Shear Z	0.6562
Moment X	1.7269
Moment Y (Twist)	0.0747
Moment Z	11.9831

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.2561
Shear X	-0.2078
Shear Z	0.3800
Moment X	0.9962
Moment Y (Twist)	0.0478
Moment Z	6.8808

## 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	-0.0060	2.5569	-0.0116	-0.0308	0.0023	0.0789
ULS: 2. D + L	-0.0060	2.5569	-0.0116	-0.0308	0.0023	0.0789
ULS: 3. D + (S or Lr or R)	-0.0298	10.9032	-0.0576	-0.1532	0.0115	0.2765
ULS: 3. D + (S or Lr or R)	-0.0060	2.5569	-0.0116	-0.0308	0.0023	0.0789
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0239	8.8166	-0.0461	-0.1226	0.0092	0.2271

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0060	2.5569	-0.0116	-0.0308	0.0023	0.0789
ULS: 5b. D + 0.7E	-0.0060	2.5569	-0.0116	-0.0308	0.0023	0.0789
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0239	8.8166	-0.0461	-0.1226	0.0092	0.2271
ULS: 8. 0.6D + 0.7E	-0.0036	1.5341	-0.0070	-0.0185	0.0014	0.0473
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2821	5.7890	-0.0286	-0.0762	0.0033	3.0831
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2821	5.7890	-0.0286	-0.0762	0.0033	3.0831
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0774	1.6829	-0.0054	-0.0146	-0.0018	2.4748
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1519	0.5084	-0.0039	-0.0095	0.0091	-8.1207
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2309	11.2407	-0.0588	-0.1567	0.0100	2.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2309	11.2407	-0.0588	-0.1567	0.0100	2.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0387	8.1611	-0.0414	-0.1105	0.0061	2.0240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0946	7.2802	-0.0403	-0.1067	0.0143	-5.9226
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2130	4.9810	-0.0243	-0.0648	0.0031	2.3321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2130	4.9810	-0.0243	-0.0648	0.0031	2.3321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0565	1.9014	-0.0069	-0.0187	-0.0007	1.8758
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1124	1.0205	-0.0059	-0.0148	0.0074	-6.0708
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2796	4.7662	-0.0239	-0.0639	0.0024	3.0516
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2796	4.7662	-0.0239	-0.0639	0.0024	3.0516
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0798	0.6601	-0.0007	-0.0023	-0.0027	2.4432
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1543	-0.5144	0.0007	0.0028	0.0082	-8.1522

### 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	19.1162
Shear X	-0.4784
Shear Z	-0.1020
Moment X	-0.2731
Moment Y (Twist)	0.0244
Moment Z	14.0832

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	11.2407
Shear X	-0.2821
Shear Z	-0.0588
Moment X	-0.1567
Moment Y (Twist)	0.0143
Moment Z	8.1522

### 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	-0.0060	2.5569	0.0116	0.0308	-0.0023	0.0789
ULS: 2. D + L	-0.0060	2.5569	0.0116	0.0308	-0.0023	0.0789
ULS: 3. D + (S or Lr or R)	-0.0298	10.9032	0.0576	0.1532	-0.0115	0.2765
ULS: 3. D + (S or Lr or R)	-0.0060	2.5569	0.0116	0.0308	-0.0023	0.0789
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0239	8.8166	0.0461	0.1226	-0.0092	0.2271
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0060	2.5569	0.0116	0.0308	-0.0023	0.0789
ULS: 5b. D + 0.7E	-0.0060	2.5569	0.0116	0.0308	-0.0023	0.0789
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0239	8.8166	0.0461	0.1226	-0.0092	0.2271
ULS: 8. 0.6D + 0.7E	-0.0036	1.5341	0.0070	0.0185	-0.0014	0.0473
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2821	5.7890	0.0286	0.0762	-0.0033	3.0831
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2821	5.7890	0.0286	0.0762	-0.0033	3.0831
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0774	1.6829	0.0054	0.0146	0.0018	2.4748
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1519	0.5084	0.0039	0.0095	-0.0091	-8.1207

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	-0.2309	11.2407	0.0588	0.1567	-0.0099	2.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2309	11.2407	0.0588	0.1567	-0.0099	2.4803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0387	8.1611	0.0414	0.1105	-0.0061	2.0240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0946	7.2802	0.0403	0.1067	-0.0142	-5.9226
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2130	4.9810	0.0243	0.0649	-0.0031	2.3321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2130	4.9810	0.0243	0.0649	-0.0031	2.3321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0565	1.9014	0.0069	0.0187	0.0007	1.8758
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1124	1.0205	0.0059	0.0148	-0.0074	-6.0708
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2796	4.7662	0.0239	0.0639	-0.0024	3.0516
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2796	4.7662	0.0239	0.0639	-0.0024	3.0516
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0798	0.6601	0.0007	0.0023	0.0027	2.4432
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1543	-0.5144	-0.0007	-0.0028	-0.0082	-8.1522

### 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	19.1162
Shear X	-0.4784
Shear Z	0.1020
Moment X	0.2731
Moment Y (Twist)	0.0242
Moment Z	14.0832

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	11.2407
Shear X	-0.2821
Shear Z	0.0588
Moment X	0.1567
Moment Y (Twist)	0.0142
Moment Z	8.1522

### 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	0.0060	1.9716	-0.0741	-0.1942	0.0067	-0.0135
ULS: 2. D + L	0.0060	1.9716	-0.0741	-0.1942	0.0067	-0.0135
ULS: 3. D + (S or Lr or R)	0.0298	8.0119	-0.3670	-0.9635	0.0332	-0.1894
ULS: 3. D + (S or Lr or R)	0.0060	1.9716	-0.0741	-0.1942	0.0067	-0.0135
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0239	6.5018	-0.2938	-0.7712	0.0266	-0.1454
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0060	1.9716	-0.0741	-0.1942	0.0067	-0.0135
ULS: 5b. D + 0.7E	0.0060	1.9716	-0.0741	-0.1942	0.0067	-0.0135
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0239	6.5018	-0.2938	-0.7712	0.0266	-0.1454
ULS: 8. 0.6D + 0.7E	0.0036	1.1829	-0.0444	-0.1165	0.0040	-0.0081
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2054	4.3107	-0.1891	-0.4941	0.0350	2.3639
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2054	4.3107	-0.1891	-0.4941	0.0350	2.3639
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0542	1.3423	-0.0464	-0.1218	0.0035	2.0592
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.1576	0.4828	0.0051	0.0111	-0.0199	-6.8808
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1347	8.2561	-0.3800	-0.9962	0.0479	1.6377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1347	8.2561	-0.3800	-0.9962	0.0479	1.6377
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0599	6.0298	-0.2730	-0.7169	0.0242	1.4091
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1375	5.3852	-0.2344	-0.6173	0.0067	-5.2959
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.1525	3.7259	-0.1603	-0.4191	0.0280	1.7696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1525	3.7259	-0.1603	-0.4191	0.0280	1.7696
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0421	1.4996	-0.0533	-0.1399	0.0043	1.5410
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1197	0.8550	-0.0147	-0.0402	-0.0133	-5.1640

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2078	3.5220	-0.1594	-0.4165	0.0324	2.3693
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2078	3.5220	-0.1594	-0.4165	0.0324	2.3693
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0517	0.5536	-0.0168	-0.0441	0.0008	2.0646
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.1552	-0.3058	0.0347	0.0887	-0.0226	-6.8754

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.9791
Shear X	-0.3523
Shear Z	-0.6562
Moment X	-1.7269
Moment Y (Twist)	0.0750
Moment Z	11.9832

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.2561
Shear X	-0.2078
Shear Z	-0.3800
Moment X	-0.9962
Moment Y (Twist)	0.0479
Moment Z	6.8808

## Project Details

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

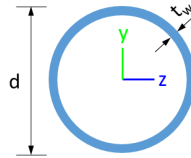


## Design Input Information

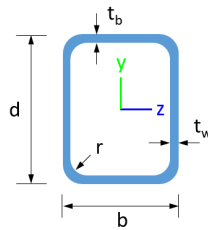
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

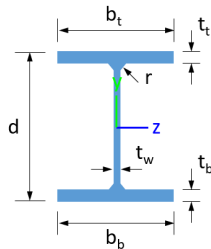
### Section Dimensions



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



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



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

### Section Properties

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

3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties									
Member ID	Section ID	K <sub>z</sub> L (ft)	K <sub>y</sub> L (ft)	L <sub>b</sub> (ft)	C <sub>b</sub>	LS T	LS C	L D	
1	7	18.34	18.34	8.74	-	30	20	0	1
2	6	1.30	1.30	2.00	-	30	20	0	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.58,1.18,1.18,1.19,1.16,1.18,1.18,1.18,1.18,1.18,1.18,1.1	30	20	0	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.69,1.68,1.68,1.68,1.44,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	30	20	0	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.12,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.6	30	20	0	1
6	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.19,1.05,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.19,1.1	30	20	0	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.54,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	30	20	0	1
8	20	1.33	1.33	2.05	1.27,1.27,1.27,1.27,1.27,1.27,1.26,1.26,1.22,1.35,1.26,1.26,2.27,1.58,1.26,1.26,1.26,1.28,1.26,1.26,1.1	30	20	0	1
9	3	2.60	2.60	4.00	-	30	20	0	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.67,1.67,1.51,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	30	20	0	1
11	20	1.33	1.33	2.05	1.27,1.27,1.27,1.27,1.27,1.27,1.28,1.28,1.31,1.21,1.28,1.28,1.37,1.43,1.28,1.28,1.28,1.25,1.28,1.28,1.3	30	20	0	1
12	6	1.30	1.30	2.00	-	30	20	0	1
13	20	4.88	4.00	7.50	1.27,1.28,1.27,1.28,1.27,1.27,1.27,1.27,1.28,2.94,1.27,1.27,1.27,1.28,1.28,1.28,1.28,1.28,1.27,1.27,1.2	30	20	0	1
14	20	4.88	4.00	7.50	1.27,1.27,1.27,1.27,1.27,1.27,1.28,1.28,1.25,1.29,1.27,1.27,2.32,1.40,1.27,1.27,1.27,1.28,1.27,1.27,1.2	30	20	0	1
15	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.27,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	30	20	0	1
16	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	30	20	0	1
101	7	18.34	18.34	8.74	-	30	20	0	1
102	6	1.30	1.30	2.00	-	30	20	0	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.26,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1	30	20	0	1
104	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.43,1.69,1.67,1.67,1.67,1.67,1.67,1.68,1.68,1.7	30	20	0	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.75,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	30	20	0	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.33,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1	30	20	0	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,2.34,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	30	20	0	1
108	20	1.33	1.33	2.05	2.33,2.35,2.33,2.36,2.34,2.33,2.37,2.37,2.25,1.97,2.36,2.36,1.18,1.42,2.39,2.39,2.37,2.11,2.37,2.37,1.9	30	20	0	1
109	3	2.60	2.60	4.00	-	30	20	0	1
110	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.50,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	30	20	0	1
111	20	1.33	1.33	2.05	2.23,2.24,2.23,2.24,2.24,2.23,2.11,2.11,2.08,1.00,2.11,2.11,1.83,1.60,2.18,2.18,2.12,2.33,2.11,2.11,2.0	30	20	0	1
112	6	1.30	1.30	2.00	-	30	20	0	1

113	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.01,1.06,1.06,1.08,1.16,1.06,1.06,1.06,1.05,1.06,1.06,1.0	300	200	1
114	20	4.88	4.00	7.50	1.05,1.06,1.05,1.06,1.06,1.05,1.05,1.05,1.05,1.07,1.05,1.05,1.09,1.45,1.06,1.06,1.05,1.06,1.05,1.05,1.0	300	200	1
115	20	6.63	6.63	10.20	1.15,1.16,1.15,1.16,1.16,1.15,1.15,1.15,1.14,2.15,1.15,1.15,1.12,1.10,1.15,1.15,1.15,1.17,1.15,1.15,1.1	300	200	1
116	20	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.20,1.12,1.16,1.16,1.09,1.08,1.16,1.16,1.16,1.15,1.16,1.16,1.2	300	200	1
201	7	18.34	18.34	8.74	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.33,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
204	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.50,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	300	200	1
205	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,2.34,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.26,1.18,1.18,1.19,1.17,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
207	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.75,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
208	20	1.33	1.33	2.05	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.12,1.59,2.08,2.08,1.09,1.30,2.07,2.07,2.07,1.93,2.07,2.07,2.3	300	200	1
209	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.69,1.67,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.67,1.67,1.67,1.43,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	300	200	1
211	20	1.33	1.33	2.05	2.02,2.03,2.02,2.03,2.03,2.02,1.95,1.95,1.77,1.12,1.92,1.92,1.54,1.42,2.00,2.00,1.96,2.08,1.94,1.94,1.7	300	200	1
212	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.01,1.06,1.06,1.08,1.16,1.06,1.06,1.06,1.05,1.06,1.06,1.0	300	200	1
214	20	4.88	4.00	7.50	1.05,1.06,1.05,1.06,1.06,1.05,1.05,1.05,1.05,1.07,1.05,1.05,1.09,1.45,1.06,1.06,1.05,1.06,1.05,1.05,1.0	300	200	1
215	20	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.00,1.17,1.17,1.13,1.12,1.17,1.17,1.17,1.19,1.17,1.17,1.1	300	200	1
216	20	6.63	6.63	10.20	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.22,1.14,1.18,1.18,1.06,1.10,1.18,1.18,1.18,1.17,1.18,1.18,1.2	300	200	1
301	7	18.34	18.34	8.74	-	300	200	1
302	6	1.30	1.30	2.00	-	300	200	1
303	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.19,1.05,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.1	300	200	1
304	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.68,1.68,1.67,1.67,1.51,1.69,1.67,1.67,1.67,1.67,1.67,1.68,1.68,1.7	300	200	1
305	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.54,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
306	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.58,1.18,1.18,1.19,1.16,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1
307	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.12,1.67,1.67,1.68,1.65,1.67,1.67,1.67,1.67,1.67,1.6	300	200	1
308	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
309	3	2.60	2.60	4.00	-	300	200	1
310	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.69,1.68,1.68,1.68,1.44,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.7	300	200	1
311	20	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.27,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1
312	6	1.30	1.30	2.00	-	300	200	1
313	20	4.88	4.00	7.50	1.27,1.28,1.27,1.28,1.27,1.27,1.27,1.27,1.28,2.93,1.27,1.27,1.27,1.28,1.28,1.28,1.28,1.28,1.27,1.27,1.2	300	200	1
314	20	4.88	4.00	7.50	1.27,1.27,1.27,1.27,1.27,1.27,1.28,1.28,1.25,1.29,1.27,1.27,2.32,1.40,1.27,1.27,1.27,1.28,1.27,1.27,1.2	300	200	1



212	251.01	248.88	27.10	27.10	75.30	75.30
213	159.30	97.43	30.77	6.46	56.26	44.91
214	159.30	97.43	31.91	6.46	56.26	44.91
215	159.30	75.13	19.26	6.46	56.26	44.91
216	159.30	75.13	20.48	6.46	56.26	44.91
301	251.16	124.39	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	113.66	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	113.66	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	38.86	6.46	56.26	44.91
314	159.30	97.43	37.57	6.46	56.26	44.91
315	159.30	75.13	20.85	6.46	56.26	44.91
316	159.30	75.13	20.84	6.46	56.26	44.91

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.112	0.283	0.094	0.005	0.009	0.301	#16	0.490	Not Required	Pass
2	0.002	0.362	0.013	0.081	0.002	0.375	#21	0.036	Not Required	Pass
3	0.002	0.565	0.017	0.056	0.005	0.583	#21	0.046	Not Required	Pass
4	0.001	0.556	0.037	0.056	0.008	0.593	#21	0.122	Not Required	Pass
5	0.002	0.351	0.009	0.056	0.002	0.360	#21	0.076	Not Required	Pass
6	0.002	0.706	0.042	0.072	0.011	0.749	#21	0.046	Not Required	Pass
7	0.002	0.437	0.051	0.070	0.014	0.457	#21	0.076	Not Required	Pass
8	0.001	0.145	0.044	0.042	0.005	0.167	#21	0.102	Not Required	Pass
9	0.002	0.085	0.035	0.003	0.003	0.121	#21	0.206	Not Required	Pass
10	0.003	0.688	0.028	0.069	0.004	0.694	#21	0.082	Not Required	Pass
11	0.003	0.145	0.050	0.044	0.005	0.162	#21	0.102	Not Required	Pass
12	0.001	0.509	0.020	0.102	0.003	0.525	#21	0.054	Not Required	Pass
13	0.005	0.110	0.116	0.058	0.006	0.161	#21	0.306	Not Required	Pass
14	0.002	0.108	0.109	0.056	0.006	0.161	#24	0.204	Not Required	Pass
15	0.000	0.019	0.010	0.016	0.002	0.030	#21	Not Required	Not Required	Pass
16	0.000	0.019	0.010	0.016	0.002	0.030	#21	Not Required	Not Required	Pass
101	0.154	0.333	0.015	0.006	0.001	0.350	#16	0.490	Not Required	Pass
102	0.001	0.619	0.023	0.127	0.003	0.642	#21	0.054	Not Required	Pass
103	0.002	0.883	0.018	0.088	0.003	0.901	#21	0.046	Not Required	Pass
104	0.002	0.873	0.039	0.087	0.008	0.896	#21	0.082	Not Required	Pass
105	0.002	0.547	0.043	0.088	0.012	0.560	#21	0.076	Not Required	Pass
106	0.002	0.863	0.012	0.086	0.002	0.873	#21	0.046	Not Required	Pass
107	0.002	0.536	0.037	0.086	0.009	0.544	#21	0.076	Not Required	Pass
108	0.001	0.055	0.041	0.049	0.005	0.097	#21	0.102	Not Required	Pass
109	0.004	0.094	0.014	0.001	0.000	0.110	#21	0.206	Not Required	Pass
110	0.002	0.850	0.040	0.085	0.009	0.880	#21	0.082	Not Required	Pass

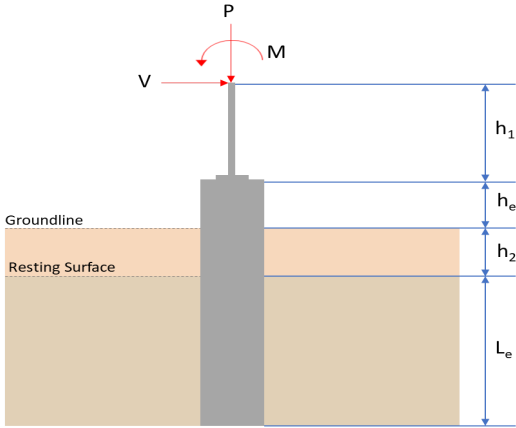
111	0.003	0.052	0.041	0.050	0.005	0.094	#21	0.102	Not Required	Pass
112	0.001	0.596	0.023	0.124	0.003	0.619	#21	0.054	Not Required	Pass
113	0.005	0.277	0.113	0.070	0.006	0.355	#21	0.306	Not Required	Pass
114	0.004	0.285	0.114	0.069	0.006	0.356	#21	0.306	Not Required	Pass
115	0.006	0.417	0.057	0.056	0.005	0.476	#21	0.507	Not Required	Pass
116	0.003	0.409	0.060	0.055	0.005	0.470	#21	0.507	Not Required	Pass
201	0.154	0.333	0.015	0.006	0.001	0.350	#16	0.490	Not Required	Pass
202	0.001	0.596	0.023	0.124	0.003	0.619	#21	0.054	Not Required	Pass
203	0.002	0.863	0.012	0.086	0.002	0.873	#21	0.046	Not Required	Pass
204	0.002	0.850	0.040	0.085	0.009	0.880	#21	0.082	Not Required	Pass
205	0.002	0.536	0.037	0.086	0.009	0.544	#21	0.076	Not Required	Pass
206	0.002	0.883	0.018	0.088	0.003	0.901	#21	0.046	Not Required	Pass
207	0.002	0.547	0.043	0.088	0.012	0.560	#21	0.076	Not Required	Pass
208	0.001	0.082	0.048	0.055	0.005	0.094	#24	0.102	Not Required	Pass
209	0.004	0.094	0.014	0.001	0.000	0.110	#21	0.206	Not Required	Pass
210	0.002	0.873	0.039	0.087	0.008	0.896	#21	0.082	Not Required	Pass
211	0.003	0.086	0.047	0.056	0.005	0.097	#21	0.102	Not Required	Pass
212	0.001	0.619	0.023	0.127	0.003	0.642	#21	0.054	Not Required	Pass
213	0.005	0.277	0.113	0.070	0.006	0.355	#21	0.306	Not Required	Pass
214	0.004	0.285	0.114	0.069	0.006	0.356	#21	0.306	Not Required	Pass
215	0.005	0.270	0.058	0.050	0.005	0.330	#21	0.507	Not Required	Pass
216	0.002	0.256	0.058	0.049	0.005	0.314	#21	0.507	Not Required	Pass
301	0.112	0.283	0.094	0.005	0.009	0.301	#16	0.490	Not Required	Pass
302	0.001	0.509	0.020	0.102	0.003	0.525	#21	0.054	Not Required	Pass
303	0.002	0.706	0.042	0.072	0.011	0.749	#21	0.046	Not Required	Pass
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314	0.002	0.108	0.109	0.056	0.006	0.161	#24	0.306	Not Required	Pass
315	0.006	0.446	0.057	0.044	0.005	0.503	#21	0.507	Not Required	Pass
316	0.003	0.440	0.059	0.042	0.005	0.500	#21	0.507	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
$F_y$	Specified minimum yield stress
$F_u$	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
$I_{yp}$	Moment of inertia about the Y axes
$I_{zp}$	Moment of inertia about the Z axes
$I_w$	Warping constant
$S_{yp}$	Plastic section modulus about the Y axis
$S_{zp}$	Plastic section modulus about the Z axis

KL	Effective length
$C_b$	Buckling modification factor (from all load combinations)
$L_b$	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
$P_n$	Nominal axial strength (tension/compression)
$M_n$	Nominal flexural strength (about Z/Y axis)
$V_n$	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
$M_z$	Design ratio in case of bending about Z axis
$M_y$	Design ratio in case of bending about Y axis
$V_y$	Design ratio in case of shear along Y axis
$V_z$	Design ratio in case of shear along Z axis
(P, $M_z$ , $M_y$ )	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS																										
	<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: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 4.5</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 1088 1225 1189"> <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 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.256</td> <td>13.979</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.208</td> <td>-0.352</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.380</td> <td>0.656</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.996</td> <td>1.727</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>6.881</td> <td>11.983</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)	8.256	13.979	$V_x$ (kip)	-0.208	-0.352	$V_z$ (kip)	0.380	0.656	$M_x$ (kipft)	0.996	1.727	$M_z$ (kipft)	6.881	11.983	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.208 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.033121 \text{ kip/ft}$																											

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(6.881 \text{ kipft}) + ((-0.208 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0957 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2931 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.38 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.06051 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.996 \text{ kipft}) + ((0.38 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1586 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.8445 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.2931 \text{ ft}), (2.8445 \text{ ft})]$$

$$L_{e,req} = 4.293 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.293 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.954$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(8.256 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.516 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.516 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.258$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.033121 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.0957 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.0957 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.0957 \text{ kipft/ft})) + (4 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0312 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.0957 \text{ kipft/ft})) + (3 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.0957 \text{ kipft/ft})) + (2 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.19193 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.0957 \text{ kipft/ft})) + ((-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.60514 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0312 \text{ ft})}{2}$$

$$p_a = 0.22734 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19193 \text{ kip/ft}^2)}{(0.22734 \text{ kip/ft}^2)}$$

$$Ratio = 0.84425$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.60514 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.89651$$

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

Status: **PASS**  
Ratio: **0.900**

#### Considering z-direction:

$H_o = 0.06051 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.1586 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.1586 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.06051 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.1586 \text{ kipft/ft})) + (4 \times (0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.2001 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.1586 \text{ kipft/ft})) + (3 \times (0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.1586 \text{ kipft/ft})) + (2 \times (0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.076449 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.1586 \text{ kipft/ft})) + ((0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.17466 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2001 \text{ ft})}{2}$$

$$p_a = 0.24001 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.076449 \text{ kip/ft}^2)}{(0.24001 \text{ kip/ft}^2)}$$

$$Ratio = 0.31852$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.17466 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.25876$$

Status: **PASS**  
Ratio: **0.320**

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.352 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.056051 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(11.983 \text{ kipft}) + ((-0.352 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9081 \text{ kipft/ft}$$

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

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

$$E = \frac{(1.9081 \text{ kipft/ft})}{(-0.056051 \text{ kip/ft})}$$

$$E = 34.043 \text{ ft}$$

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

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

$$a = \frac{(4 \times (1.9081 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.9081 \text{ kipft/ft})) + (4 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.056051 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (1.9081 \text{ kip/ft})) + (4 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0304 \text{ ft}$$

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

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

$$V_{max} = ((-0.056051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.0304 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.0304 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.1575 \text{ kip}$$

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

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

$$M_{max} = ((-0.056051 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[ \left( \frac{(34.043 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0304 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.0304 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^3 \right] + \left[ \left( \frac{3 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.0304 \text{ ft})}{(2 \times (4.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 7.0115 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.656 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10446 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.727 \text{ kipft}) + ((0.656 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.275 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.275 \text{ kipft/ft})}{(0.10446 \text{ kip/ft})}$$

$$E = 2.6326 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.275 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.10446 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.275 \text{ kipft/ft})) + (4 \times (0.10446 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1997 \text{ ft}$$

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

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

$$V_{max} = ((0.10446 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.1997 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.1997 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.71029 \text{ kip}$$

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

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

$$M_{max} = ((0.10446 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[ \left( \frac{(2.6326 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.4301 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.8$  - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.979 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.132 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.132 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p>Ratio - Capacity</p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(13.979 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0052254</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.979 \text{ kip} \rightarrow 13979 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13979 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.35 \text{ kip}$$

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

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.35 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.35 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3  $V_{s,b}$  - Shear strength of steel (b)

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

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

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

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.35 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.31 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 3.1575 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.1575 \text{ kip})}{(111.31 \text{ kip})}$$

$$Ratio = 0.028367$$

**Considering z-direction:**

$V_{max} = 0.71029 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.71029 \text{ kip})}{(111.31 \text{ kip})}$$

$$Ratio = 0.0063813$$

Status: **PASS**  
 Ratio: **0.030**

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 7.0115 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(7.0115 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.028091$$

Status: **PASS**  
 Ratio: **0.030**

**Considering z-direction:**

$M_{max} = 1.4301 \text{ kipft}$  - Maximum moment in the z-direction,

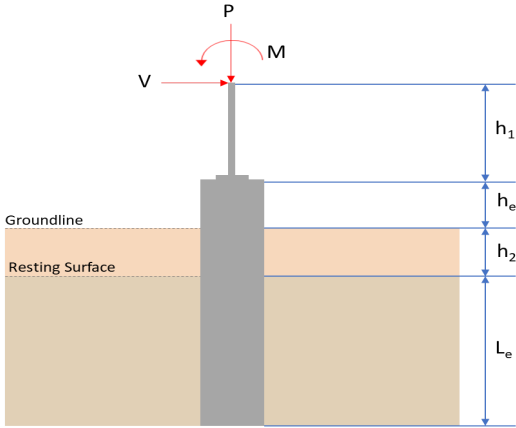
*Ratio* - Capacity

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

$$Ratio = \frac{(1.4301 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0057294$$

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: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 4.5</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 1088 1225 1189"> <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 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.256</td> <td>13.979</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.208</td> <td>-0.352</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.380</td> <td>-0.656</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.996</td> <td>-1.727</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>6.881</td> <td>11.983</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)	8.256	13.979	$V_x$ (kip)	-0.208	-0.352	$V_z$ (kip)	-0.380	-0.656	$M_x$ (kipft)	-0.996	-1.727	$M_z$ (kipft)	6.881	11.983	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
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$M_x$ (kipft)	-0.996	-1.727																										
$M_z$ (kipft)	6.881	11.983																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.208 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.033121 \text{ kip/ft}$																											

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(6.881 \text{ kipft}) + ((-0.208 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.0957 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.2931 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.38 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.06051 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.996 \text{ kipft}) + ((-0.38 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1586 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8238 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(4.2931 \text{ ft}), (1.8238 \text{ ft})]$$

$$L_{e,req} = 4.293 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(4.293 \text{ ft})}{(4.5 \text{ ft})}$$

$$Ratio = 0.954$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(8.256 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.516 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.516 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.258$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.033121 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.0957 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.0957 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.0957 \text{ kipft/ft})) + (4 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0312 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.0957 \text{ kipft/ft})) + (3 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (1.0957 \text{ kipft/ft})) + (2 \times (-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.19193 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.0957 \text{ kipft/ft})) + ((-0.033121 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.60514 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.0312 \text{ ft})}{2}$$

$$p_a = 0.22734 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19193 \text{ kip/ft}^2)}{(0.22734 \text{ kip/ft}^2)}$$

$$Ratio = 0.84425$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.60514 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.89651$$

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

Status: **PASS**  
Ratio: **0.900**

#### Considering z-direction:

$H_o = -0.06051 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.1586 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.1586 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.06051 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.1586 \text{ kipft/ft})) + (4 \times (-0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.2001 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.1586 \text{ kipft/ft})) + (3 \times (-0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.1586 \text{ kipft/ft})) + (2 \times (-0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = -0.017929 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.1586 \text{ kipft/ft})) + ((-0.06051 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.013305 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2001 \text{ ft})}{2}$$

$$p_a = 0.24001 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.017929 \text{ kip/ft}^2)}{(0.24001 \text{ kip/ft}^2)}$$

$$Ratio = -0.074702$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 0.675 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.013305 \text{ kip/ft}^2)}{(0.675 \text{ kip/ft}^2)}$$

$$Ratio = 0.019711$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.352 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.056051 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(11.983 \text{ kipft}) + ((-0.352 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9081 \text{ kipft/ft}$$

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

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

$$E = \frac{(1.9081 \text{ kipft/ft})}{(-0.056051 \text{ kip/ft})}$$

$$E = 34.043 \text{ ft}$$

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

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

$$a = \frac{(4 \times (1.9081 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (1.9081 \text{ kipft/ft})) + (4 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = \frac{(-0.056051 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (1.9081 \text{ kipft/ft})) + (4 \times (-0.056051 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0304 \text{ ft}$$

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

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

$$V_{max} = ((-0.056051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.0304 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.0304 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.1575 \text{ kip}$$

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

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

$$M_{max} = ((-0.056051 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[ \left( \frac{(34.043 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0304 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.0304 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (34.043 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.0304 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 7.0115 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.656 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10446 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.727 \text{ kipft}) + ((-0.656 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.275 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.275 \text{ kipft/ft})}{(-0.10446 \text{ kip/ft})}$$

$$E = 2.6326 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.275 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.10446 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.275 \text{ kipft/ft})) + (4 \times (-0.10446 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1997 \text{ ft}$$

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

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

$$V_{max} = ((-0.10446 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.1997 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.1997 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.71029 \text{ kip}$$

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

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

$$M_{max} = ((-0.10446 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[ \left( \frac{(2.6326 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left( \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.6326 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left( \frac{(3.1997 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.4301 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.8$  - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.979 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.132 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.132 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.979 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0052254$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 13.979 \text{ kip} \rightarrow 13979 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13979 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.35 \text{ kip}$$

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

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.35 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.35 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3  $V_{s,b}$  - Shear strength of steel (b)

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

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

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

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.35 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.31 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 3.1575 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.1575 \text{ kip})}{(111.31 \text{ kip})}$$

$$Ratio = 0.028367$$

**Considering z-direction:**

$V_{max} = 0.71029 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.71029 \text{ kip})}{(111.31 \text{ kip})}$$

$$Ratio = 0.0063813$$

Status: **PASS**  
 Ratio: **0.030**

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 7.0115 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(7.0115 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.028091$$

Status: **PASS**  
 Ratio: **0.030**

**Considering z-direction:**

$M_{max} = 1.4301 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(1.4301 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0057294$$

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

REFERENCES	CALCULATIONS	RESULTS
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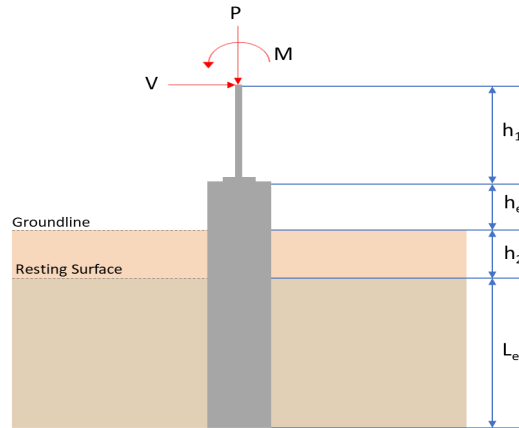
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 4.75$  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)	11.241	19.116
$V_x$ (kip)	-0.282	-0.478
$V_z$ (kip)	-0.059	-0.102
$M_x$ (kipft)	-0.157	-0.273
$M_z$ (kipft)	8.152	14.083

### Material Properties

$f'_{ck} = 2.5$  ksi - Concrete strength.

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.282 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(8.152 \text{ kipft}) + ((-0.282 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.2981 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.5094 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.059 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.009395 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.157 \text{ kipft}) + ((-0.059 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1116 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(4.5094 \text{ ft}), (1.1116 \text{ ft})]$$

$$L_{e,req} = 4.509 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$Ratio = \frac{(4.509 \text{ ft})}{(4.75 \text{ ft})}$$

$$Ratio = 0.94926$$

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

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(11.241 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.70256 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.70256 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35128$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.044904 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.2981 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.2981 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.2981 \text{ kipft/ft})) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2057 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.2981 \text{ kipft/ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.2981 \text{ kipft/ft})) + (2 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.19867 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.2981 \text{ kipft/ft})) + ((-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.63368 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2057 \text{ ft})}{2}$$

$$p_a = 0.24043 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19867 \text{ kip/ft}^2)}{(0.24043 \text{ kip/ft}^2)}$$

$$Ratio = 0.8263$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.63368 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.88937$$

Status: **PASS**  
Ratio: **0.830**

Status: **PASS**  
Ratio: **0.890**

#### Considering z-direction:

$H_o = -0.009395 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.025 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.025 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.009395 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.025 \text{ kipft/ft})) + (4 \times (-0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3818 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.025 \text{ kipft/ft})) + (3 \times (-0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (0.025 \text{ kipft/ft})) + (2 \times (-0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = -0.0026769 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.025 \text{ kipft/ft})) + ((-0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.0014292 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3818 \text{ ft})}{2}$$

$$p_a = 0.25363 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0026769 \text{ kip/ft}^2)}{(0.25363 \text{ kip/ft}^2)}$$

$$Ratio = -0.010554$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0014292 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0020058$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.478 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.076115 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(14.083 \text{ kipft}) + ((-0.478 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.2425 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.2425 \text{ kipft/ft})}{(-0.076115 \text{ kip/ft})}$$

$$E = 29.462 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.2425 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.2425 \text{ kipft/ft})) + (4 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = \frac{(-0.076115 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.2425 \text{ kip/ft})) + (4 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2051 \text{ ft}$$

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

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

$$V_{max} = ((-0.076115 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.2051 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.2051 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5506 \text{ kip}$$

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

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

$$M_{max} = ((-0.076115 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(29.462 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.2996 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.102 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.016242 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.273 \text{ kipft}) + ((-0.102 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.043471 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.043471 \text{ kipft/ft})}{(-0.016242 \text{ kip/ft})}$$

$$E = 2.6765 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.043471 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.016242 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.043471 \text{ kipft/ft})) + (4 \times (-0.016242 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3812 \text{ ft}$$

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

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

$$V_{max} = ((-0.016242 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3812 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3812 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10799 \text{ kip}$$

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

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

$$M_{max} = ((-0.016242 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(2.6765 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.2289 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.8$  - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(19.116 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.961 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.961 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(19.116 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0071457$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 19.116 \text{ kip} \rightarrow 19116 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(19116 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.03 \text{ kip}$$

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

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.03 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.03 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3  $V_{s,b}$  - Shear strength of steel (b)

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

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

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

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((121.03 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.75 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 3.5506 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.5506 \text{ kip})}{(111.75 \text{ kip})}$$

$$Ratio = 0.031771$$

Status: **PASS**  
Ratio: **0.030**

**Considering z-direction:**

$V_{max} = 0.10799 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.10799 \text{ kip})}{(111.75 \text{ kip})}$$

$$Ratio = 0.00096629$$

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 8.2996 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.2996 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.033252$$

Status: **PASS**  
Ratio: **0.030**

**Considering z-direction:**

$M_{max} = 0.2289 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.2289 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00091705$$

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

REFERENCES	CALCULATIONS	RESULTS
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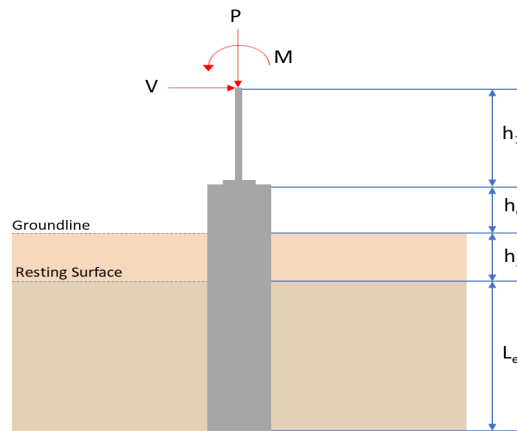
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 4.75$  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)	11.241	19.116
$V_x$ (kip)	-0.282	-0.478
$V_z$ (kip)	0.059	0.102
$M_x$ (kipft)	0.157	0.273
$M_z$ (kipft)	8.152	14.083

### Material Properties

$f'_{ck} = 2.5$  ksi - Concrete strength.

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.282 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(8.152 \text{ kipft}) + ((-0.282 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.2981 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.5094 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.059 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.009395 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.157 \text{ kipft}) + ((0.059 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4084 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.5094 \text{ ft}), (1.4084 \text{ ft})]$$

$$L_{e,req} = 4.509 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.509 \text{ ft})}{(4.75 \text{ ft})}$$

$$\text{Ratio} = 0.94926$$

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

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(11.241 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.70256 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.70256 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.35128$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.044904 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.2981 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.2981 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.2981 \text{ kipft/ft})) + (4 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2057 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.2981 \text{ kipft/ft})) + (3 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.2981 \text{ kipft/ft})) + (2 \times (-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.19867 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.2981 \text{ kipft/ft})) + ((-0.044904 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.63368 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2057 \text{ ft})}{2}$$

$$p_a = 0.24043 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19867 \text{ kip/ft}^2)}{(0.24043 \text{ kip/ft}^2)}$$

$$Ratio = 0.8263$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.63368 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.88937$$

Status: **PASS**  
Ratio: **0.830**

Status: **PASS**  
Ratio: **0.890**

#### Considering z-direction:

$H_o = 0.009395 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.025 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.025 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.009395 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.025 \text{ kipft/ft})) + (4 \times (0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3818 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.025 \text{ kipft/ft})) + (3 \times (0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (0.025 \text{ kipft/ft})) + (2 \times (0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.01107 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.025 \text{ kipft/ft})) + ((0.009395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.025164 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3818 \text{ ft})}{2}$$

$$p_a = 0.25363 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.01107 \text{ kip/ft}^2)}{(0.25363 \text{ kip/ft}^2)}$$

$$Ratio = 0.043645$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.025164 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.035317$$

Status: **PASS**  
Ratio: **0.040**

Status: **PASS**  
Ratio: **0.040**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.478 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.076115 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(14.083 \text{ kipft}) + ((-0.478 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.2425 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.2425 \text{ kipft/ft})}{(-0.076115 \text{ kip/ft})}$$

$$E = 29.462 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.2425 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{6 \times (2.2425 \text{ kipft/ft}) + 4 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft})}$$

$$a = \frac{(6 \times (2.2425 \text{ kipft/ft})) + (4 \times (-0.076115 \text{ kip/ft}) \times (4.75 \text{ ft}))}{}$$

$$a = 3.2051 \text{ ft}$$

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

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

$$V_{max} = ((-0.076115 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.2051 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.2051 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.5506 \text{ kip}$$

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

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

$$M_{max} = ((-0.076115 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(29.462 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (29.462 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.2051 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 8.2996 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.102 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.016242 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.273 \text{ kipft}) + ((0.102 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.043471 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.043471 \text{ kipft/ft})}{(0.016242 \text{ kip/ft})}$$

$$E = 2.6765 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.043471 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.016242 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.043471 \text{ kipft/ft})) + (4 \times (0.016242 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3812 \text{ ft}$$

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

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

$$V_{max} = ((0.016242 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3812 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3812 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10799 \text{ kip}$$

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

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

$$M_{max} = ((0.016242 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(2.6765 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.6765 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3812 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.2289 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,

$f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,

$\phi = 0.65$  - Reduction factor for axial strength,

$\alpha = 0.8$  - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$  - Gross area of concrete,

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(19.116 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.961 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-83.961 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(19.116 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0071457$	<p>Status: <b>PASS</b> Ratio: <b>0.010</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 19.116 \text{ kip} \rightarrow 19116 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(19116 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.03 \text{ kip}$$

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

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + (0.05 \times (2500 \text{ psi}))} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.03 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.03 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3  $V_{s,b}$  - Shear strength of steel (b)

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

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

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

$V_s$  - Governing shear strength of steel

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

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((121.03 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.75 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 3.5506 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(3.5506 \text{ kip})}{(111.75 \text{ kip})}$$

$$Ratio = 0.031771$$

**Considering z-direction:**

$V_{max} = 0.10799 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.10799 \text{ kip})}{(111.75 \text{ kip})}$$

$$Ratio = 0.00096629$$

Status: **PASS**  
Ratio: **0.030**

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 8.2996 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(8.2996 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.033252$$

Status: **PASS**  
Ratio: **0.030**

**Considering z-direction:**

$M_{max} = 0.2289 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.2289 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00091705$$

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