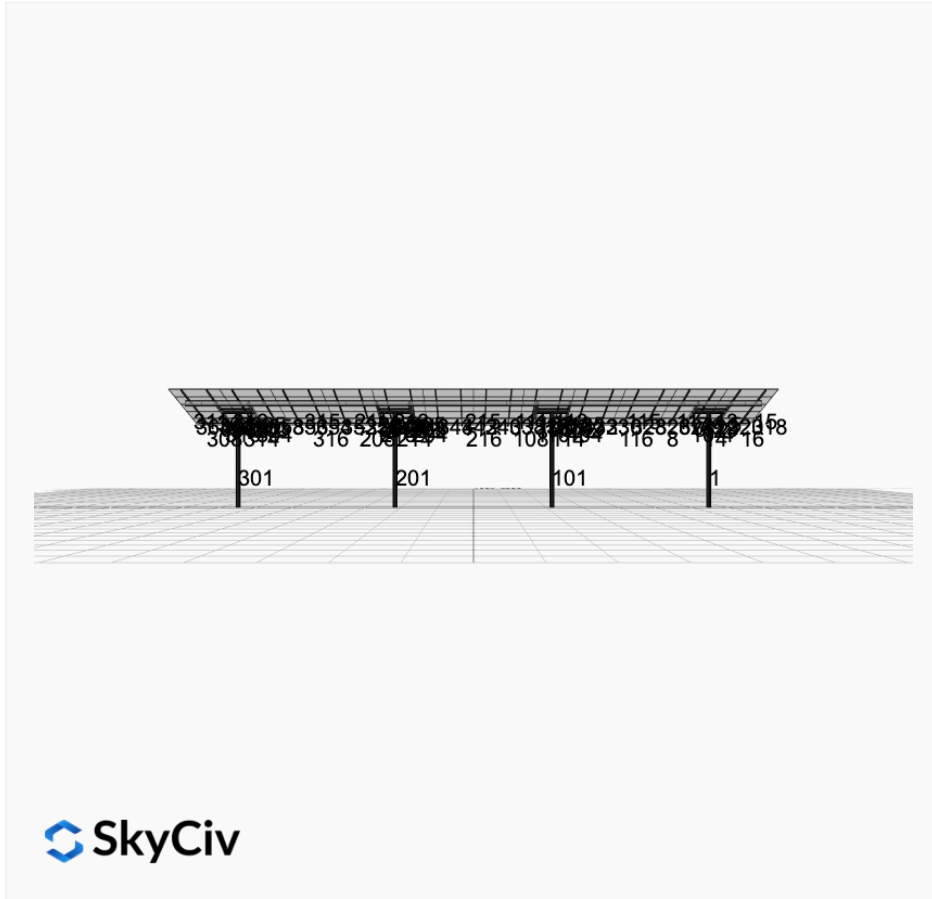


# Project Details



**Project Name:** DOT Region Office - V1Jb      **Date:** Tue Oct 29 2024  
**Location:** Smelter Ave, Great Falls, MT, USA      **Number of Modules:** 60  
**Unique ID:** 4P-19.75-6TOP-HD-24-L-5Hx12W-FF37      **Number of Poles:** 4  
**Dealer:** \_\_\_\_\_      **Date Sold:** \_\_\_\_\_



Array Dimensions N/S	15.21 ft
Array Dimensions E/W	73.00 ft
Winter Tilt Angle	15
Front Edge Clearance	10 ft

## MT Solar Bill of Materials (4P-19.75-6TOP-HD-24-L-5Hx12W-FF37)

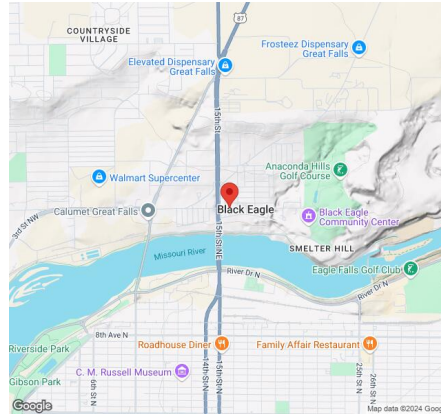
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	6
MTS-HD-Splice-57	57IN HD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

## Rail Bill of Materials

Part	Qty
Rails (180in)	24
Rail Attachment	96
Module Mid Clamp	96

<b>Part</b>	<b>Qty</b>
Module End Clamp	48
Ground Lug	12

## Site Details:



**Site Address:** Smelter Ave, Great Falls, MT, USA

### Array Specification

<b>Duty Classification:</b>	HD
<b>Module Width:</b>	36.00 in
<b>Module Length:</b>	72.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	12
<b>Total Number of Modules:</b>	60
<b>Winter Tilt Angle:</b>	15
<b>Front Edge Clearance:</b>	10
<b>Total Array Height at Tilt:</b>	13.94 ft
<b>Total Frame Length:</b>	70.75 ft
<b>Frame Weight:</b>	4209 lbs
<b>Array Dimensions N/S:</b>	15.21 ft
<b>Array Dimensions E/W:</b>	73.00 ft
<b>Rail Length:</b>	182.50 in
<b>Rail Spacing:</b>	3.04 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	11.97 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	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: 4.75 ft Pile 2: 5.00 ft Pile 3: 5.00 ft Pile 4: 4.75 ft
<b>Foundation Volume:</b>	11.556 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	Smelter Ave, Great Falls, MT, USA
<b>Wind Speed:</b>	101 mph

**Snow Load:**

32 psf

### **Design Disclaimer**

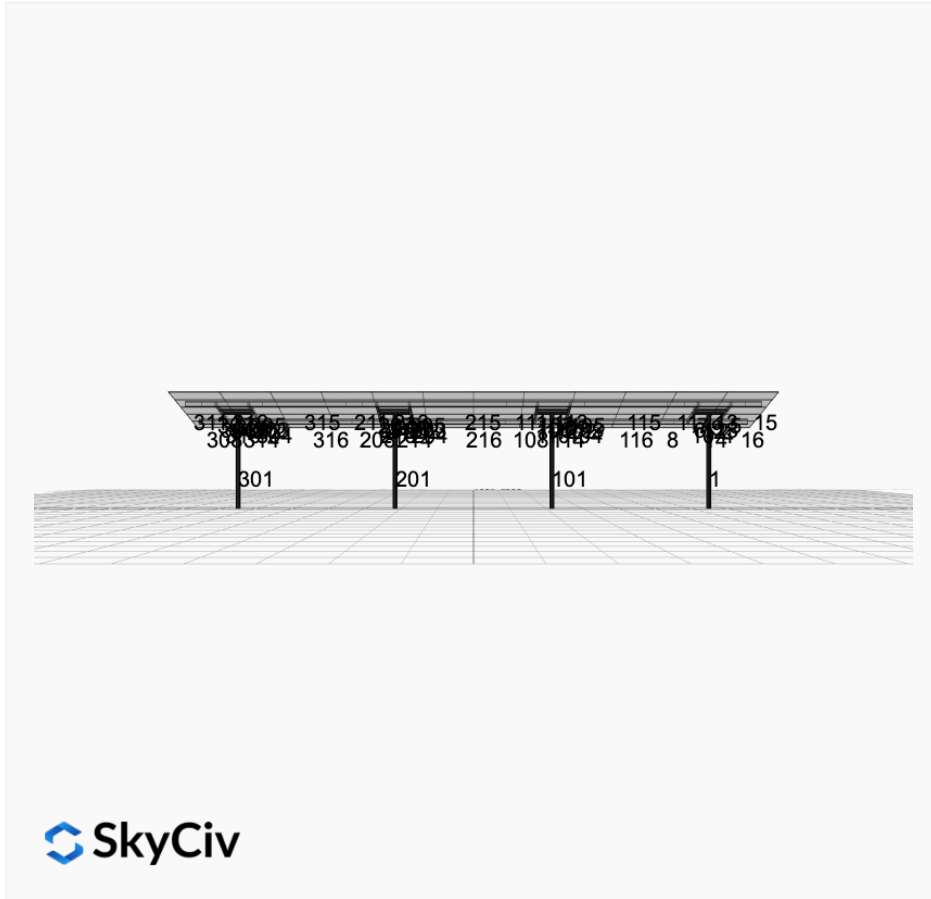
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

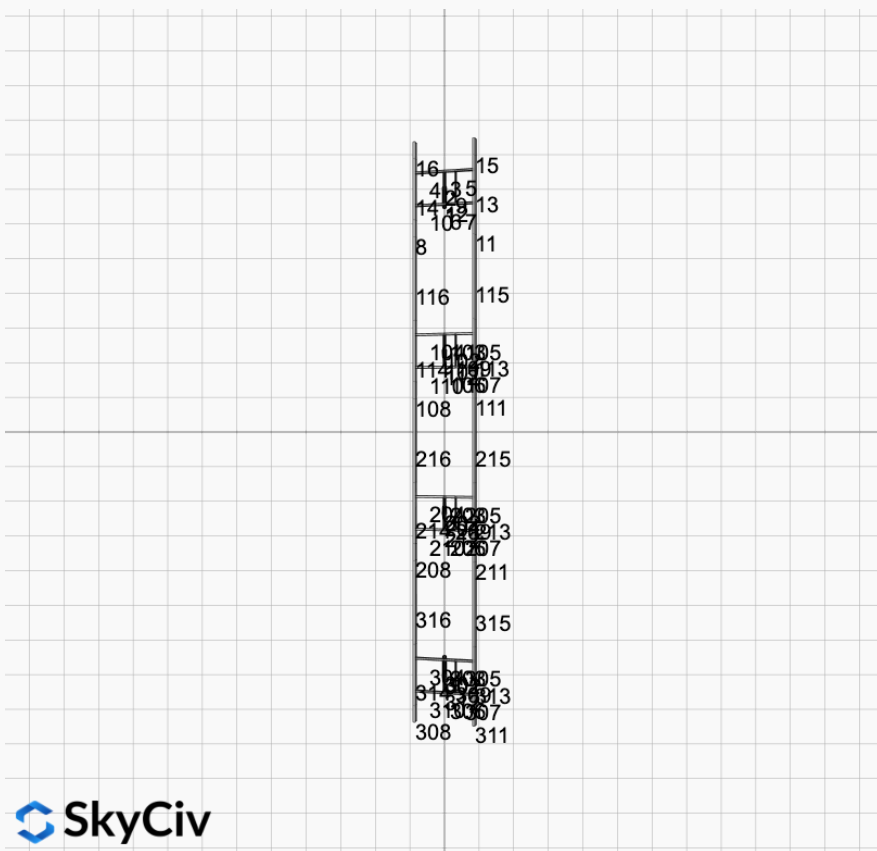
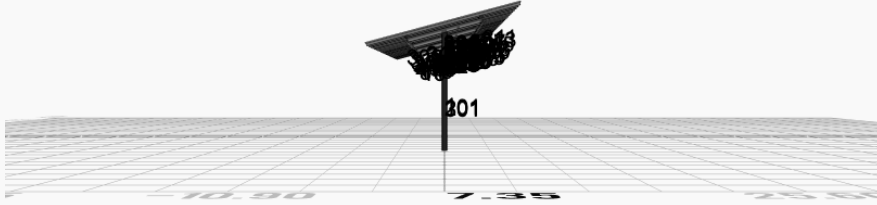
## AutoDesigner Input

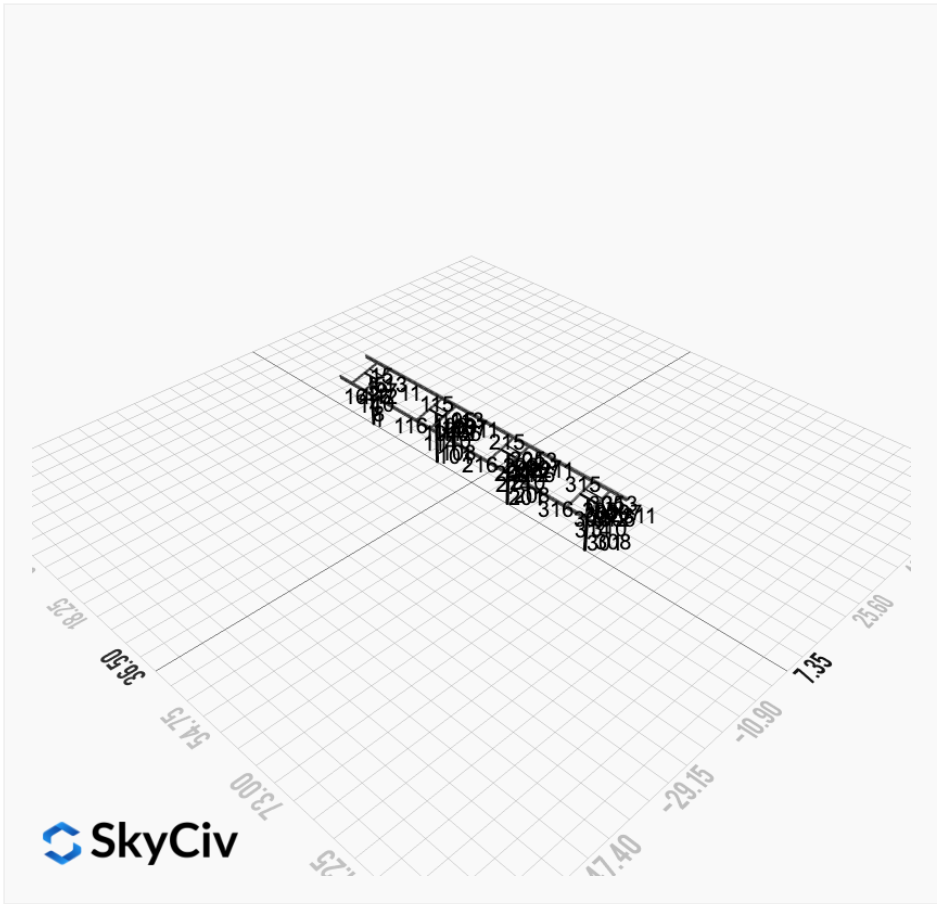
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## Design Notes:

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

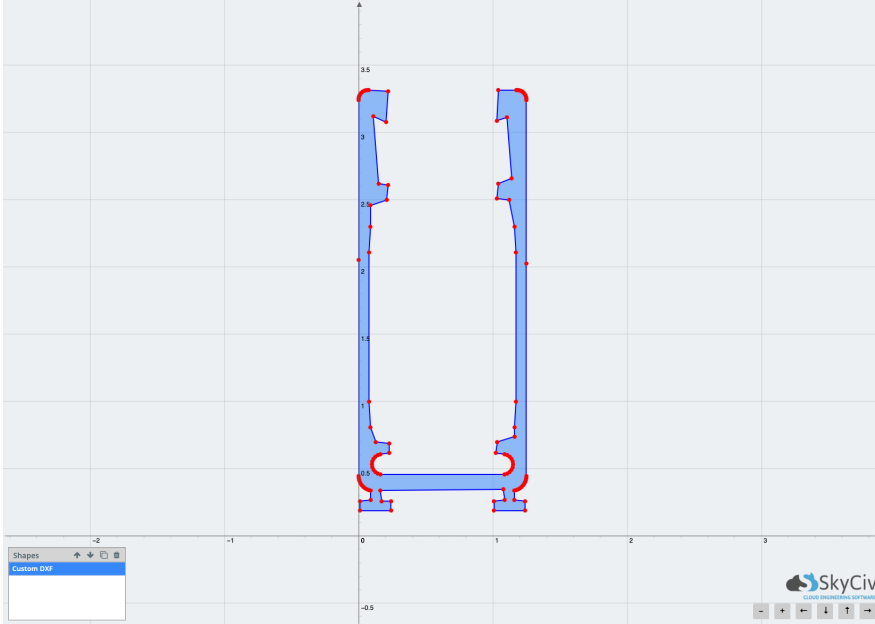






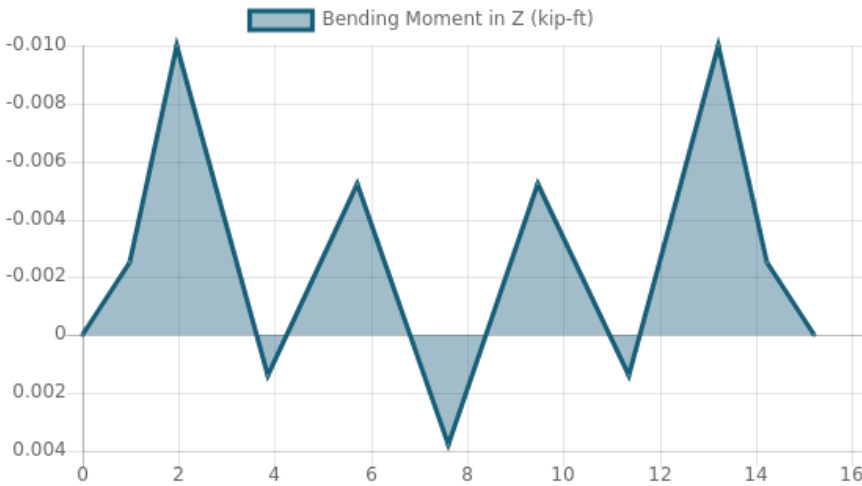
### Rail Design Check

**Rail Length:** 15.208333333333334 ft  
**Additional Restraints Required:** 4ft Spread Clamps  
**Tributary Width:** 3.0416666666666665 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Snow (X):** 0.0569 kip/ft  
**Snow (Y):** -0.0152 kip/ft  
**Wind uplift Case A:** 0.0379 kip/ft  
**Wind uplift Case A:** 0.0379 kip/ft  
**Wind uplift Case B (X):** 0.0000 kip/ft  
**Wind uplift Case B (Y):** 0.0555 kip/ft

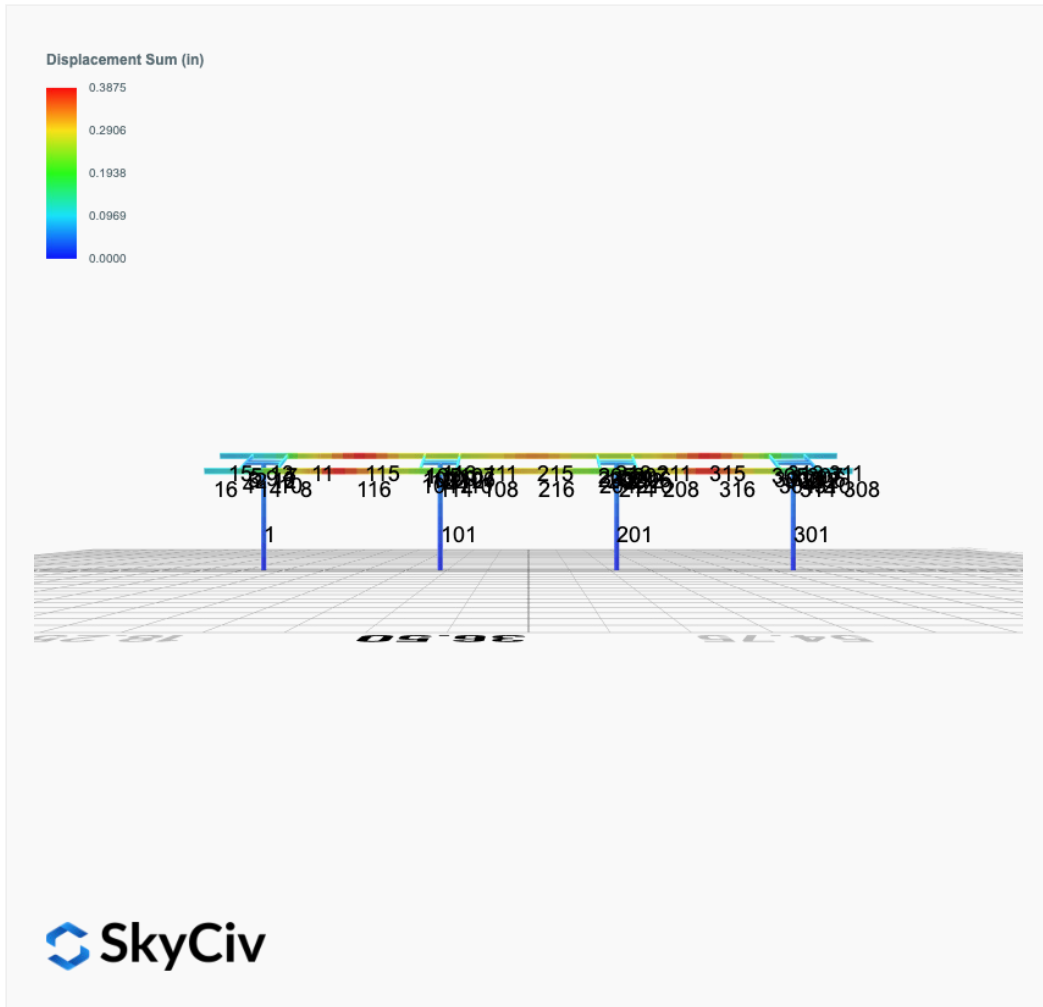


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	2.58039606	0.075	PASS
Material Yield	34.5	2.58039606	0.075	PASS
Material Strength	37	2.58039606	0.070	PASS

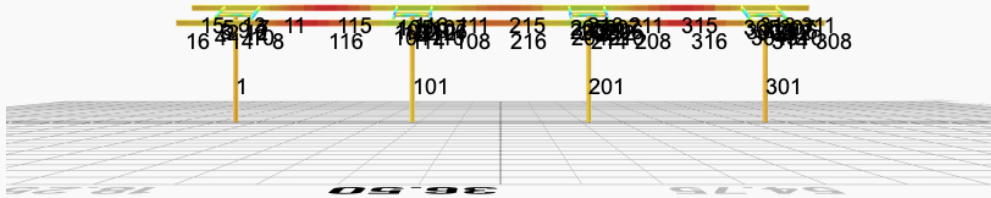
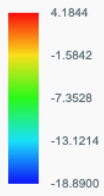
Member 1, ULS: 1. 1.4D



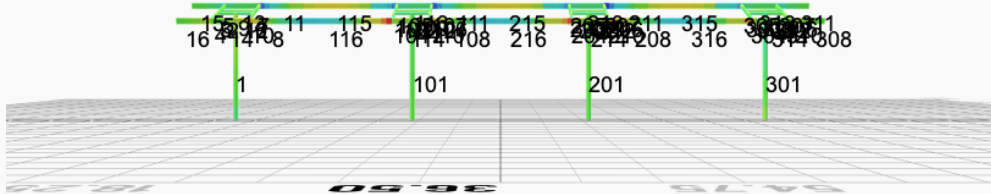
# FEM Results (Envelope Worst Case for each member)



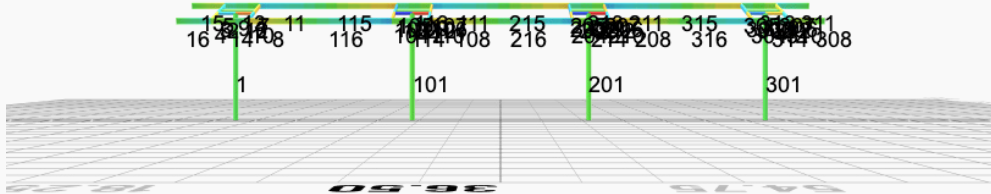
Top Bending Stress Z (ksi)



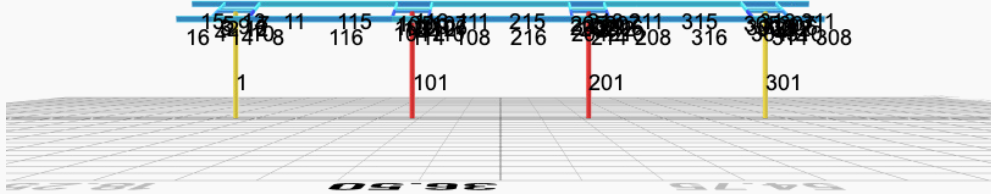
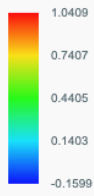
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0124	1.8300	0.0623	0.2381	-0.0228	-0.1016
ULS: 2. D + L	0.0124	1.8300	0.0623	0.2381	-0.0228	-0.1016
ULS: 3. D + (S or Lr or R)	0.0498	6.0775	0.2535	0.9710	-0.0930	-0.4907
ULS: 3. D + (S or Lr or R)	0.0124	1.8300	0.0623	0.2381	-0.0228	-0.1016
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0405	5.0157	0.2057	0.7878	-0.0754	-0.3934
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0124	1.8300	0.0623	0.2381	-0.0228	-0.1016
ULS: 5b. D + 0.7E	0.0124	1.8300	0.0623	0.2381	-0.0228	-0.1016
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0405	5.0157	0.2057	0.7878	-0.0754	-0.3934
ULS: 8. 0.6D + 0.7E	0.0074	1.0980	0.0374	0.1429	-0.0137	-0.0610
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5248	3.7206	0.1627	0.6187	-0.1106	7.4451
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5248	3.7206	0.1627	0.6187	-0.1106	7.4451
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4090	0.3989	-0.0081	-0.0278	0.0355	-3.7972
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3854	0.5855	-0.0139	-0.0492	0.0506	-9.4594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3624	6.4336	0.2810	1.0732	-0.1413	5.2666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3624	6.4336	0.2810	1.0732	-0.1413	5.2666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3379	3.9423	0.1529	0.5884	-0.0317	-3.1651
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3202	4.0823	0.1485	0.5723	-0.0204	-7.4118
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3905	3.2479	0.1376	0.5236	-0.0886	5.5584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3905	3.2479	0.1376	0.5236	-0.0886	5.5584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3099	0.7567	0.0095	0.0387	0.0209	-2.8733
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2921	0.8967	0.0051	0.0226	0.0323	-7.1200
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5297	2.9886	0.1378	0.5235	-0.1015	7.4857
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5297	2.9886	0.1378	0.5235	-0.1015	7.4857
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4041	-0.3331	-0.0330	-0.1230	0.0446	-3.7565
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3804	-0.1465	-0.0389	-0.1444	0.0597	-9.4188

### 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	10.5685
Shear X	-0.8952
Shear Z	0.4689
Moment X	1.8020
Moment Y (Twist)	0.2179
Moment Z	16.7456

### 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	6.4336
Shear X	-0.5297
Shear Z	0.2810
Moment X	1.0732
Moment Y (Twist)	0.1413
Moment Z	9.4594

## 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	-0.0124	2.3412	-0.0091	-0.0351	0.0053	0.1600
ULS: 2. D + L	-0.0124	2.3412	-0.0091	-0.0351	0.0053	0.1600
ULS: 3. D + (S or Lr or R)	-0.0498	8.1510	-0.0371	-0.1429	0.0214	0.5902
ULS: 3. D + (S or Lr or R)	-0.0124	2.3412	-0.0091	-0.0351	0.0053	0.1600
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0405	6.6986	-0.0301	-0.1159	0.0174	0.4826

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0124	2.3412	-0.0091	-0.0351	0.0053	0.1600
ULS: 5b. D + 0.7E	-0.0124	2.3412	-0.0091	-0.0351	0.0053	0.1600
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0405	6.6986	-0.0301	-0.1159	0.0174	0.4826
ULS: 8. 0.6D + 0.7E	-0.0074	1.4047	-0.0055	-0.0211	0.0032	0.0960
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6745	4.9265	-0.0152	-0.0593	-0.0021	9.3296
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6745	4.9265	-0.0152	-0.0593	-0.0021	9.3296
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5008	0.3768	-0.0014	-0.0050	0.0047	-4.4647
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4004	0.6532	-0.0110	-0.0407	0.0219	-10.7331
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5371	8.6375	-0.0346	-0.1341	0.0118	7.3599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5371	8.6375	-0.0346	-0.1341	0.0118	7.3599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3444	5.2252	-0.0243	-0.0934	0.0170	-2.9859
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2691	5.4325	-0.0315	-0.1201	0.0299	-7.6872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5090	4.2802	-0.0137	-0.0533	-0.0003	7.0372
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5090	4.2802	-0.0137	-0.0533	-0.0003	7.0372
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3725	0.8679	-0.0033	-0.0125	0.0049	-3.3085
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2972	1.0752	-0.0105	-0.0393	0.0178	-8.0098
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6696	3.9900	-0.0115	-0.0453	-0.0042	9.2656
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6696	3.9900	-0.0115	-0.0453	-0.0042	9.2656
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5058	-0.5597	0.0023	0.0090	0.0026	-4.5287
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4053	-0.2833	-0.0073	-0.0266	0.0198	-10.7971

### 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	14.2586
Shear X	-1.1323
Shear Z	-0.0596
Moment X	-0.2305
Moment Y (Twist)	0.0502
Moment Z	18.8148

### 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	8.6375
Shear X	-0.6745
Shear Z	-0.0371
Moment X	-0.1429
Moment Y (Twist)	0.0299
Moment Z	10.7971

### 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	-0.0124	2.3412	0.0091	0.0351	-0.0053	0.1600
ULS: 2. D + L	-0.0124	2.3412	0.0091	0.0351	-0.0053	0.1600
ULS: 3. D + (S or Lr or R)	-0.0498	8.1510	0.0371	0.1428	-0.0214	0.5902
ULS: 3. D + (S or Lr or R)	-0.0124	2.3412	0.0091	0.0351	-0.0053	0.1600
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0405	6.6986	0.0301	0.1159	-0.0174	0.4826
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0124	2.3412	0.0091	0.0351	-0.0053	0.1600
ULS: 5b. D + 0.7E	-0.0124	2.3412	0.0091	0.0351	-0.0053	0.1600
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0405	6.6986	0.0301	0.1159	-0.0174	0.4826
ULS: 8. 0.6D + 0.7E	-0.0074	1.4047	0.0055	0.0211	-0.0032	0.0960
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6745	4.9265	0.0152	0.0593	0.0021	9.3296
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6745	4.9265	0.0152	0.0593	0.0021	9.3296
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5008	0.3768	0.0014	0.0050	-0.0047	-4.4647
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4004	0.6532	0.0110	0.0407	-0.0219	-10.7331

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	-0.5371	8.6375	0.0346	0.1341	-0.0118	7.3598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5371	8.6375	0.0346	0.1341	-0.0118	7.3598
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3444	5.2252	0.0243	0.0933	-0.0169	-2.9859
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2691	5.4325	0.0315	0.1201	-0.0298	-7.6872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5090	4.2802	0.0137	0.0533	0.0003	7.0372
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5090	4.2802	0.0137	0.0533	0.0003	7.0372
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3725	0.8679	0.0033	0.0125	-0.0049	-3.3085
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2972	1.0752	0.0105	0.0393	-0.0178	-8.0098
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6696	3.9900	0.0115	0.0453	0.0042	9.2656
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6696	3.9900	0.0115	0.0453	0.0042	9.2656
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5058	-0.5597	-0.0023	-0.0090	-0.0026	-4.5287
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4053	-0.2833	0.0073	0.0266	-0.0198	-10.7971

### 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	14.2586
Shear X	-1.1323
Shear Z	0.0596
Moment X	0.2305
Moment Y (Twist)	0.0499
Moment Z	18.8148

### 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	8.6375
Shear X	-0.6745
Shear Z	0.0371
Moment X	0.1428
Moment Y (Twist)	0.0298
Moment Z	10.7971

### 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	0.0124	1.8300	-0.0623	-0.2381	0.0228	-0.1016
ULS: 2. D + L	0.0124	1.8300	-0.0623	-0.2381	0.0228	-0.1016
ULS: 3. D + (S or Lr or R)	0.0498	6.0775	-0.2535	-0.9710	0.0930	-0.4906
ULS: 3. D + (S or Lr or R)	0.0124	1.8300	-0.0623	-0.2381	0.0228	-0.1016
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0405	5.0157	-0.2057	-0.7878	0.0755	-0.3934
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0124	1.8300	-0.0623	-0.2381	0.0228	-0.1016
ULS: 5b. D + 0.7E	0.0124	1.8300	-0.0623	-0.2381	0.0228	-0.1016
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0405	5.0157	-0.2057	-0.7878	0.0755	-0.3934
ULS: 8. 0.6D + 0.7E	0.0074	1.0980	-0.0374	-0.1429	0.0137	-0.0610
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5248	3.7206	-0.1627	-0.6187	0.1106	7.4451
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5248	3.7206	-0.1627	-0.6187	0.1106	7.4451
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4090	0.3989	0.0081	0.0278	-0.0355	-3.7972
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3854	0.5855	0.0139	0.0492	-0.0506	-9.4594
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3624	6.4336	-0.2810	-1.0733	0.1413	5.2666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3624	6.4336	-0.2810	-1.0733	0.1413	5.2666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3379	3.9423	-0.1529	-0.5884	0.0318	-3.1651
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3202	4.0823	-0.1485	-0.5723	0.0205	-7.4117
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3905	3.2479	-0.1376	-0.5236	0.0886	5.5584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3905	3.2479	-0.1376	-0.5236	0.0886	5.5584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3099	0.7567	-0.0095	-0.0387	-0.0209	-2.8733
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2921	0.8967	-0.0051	-0.0226	-0.0323	-7.1199

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5297	2.9886	-0.1378	-0.5235	0.1015	7.4857
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5297	2.9886	-0.1378	-0.5235	0.1015	7.4857
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4041	-0.3331	0.0330	0.1230	-0.0446	-3.7565
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3804	-0.1465	0.0389	0.1444	-0.0597	-9.4188

### 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	10.5685
Shear X	-0.8952
Shear Z	-0.4689
Moment X	-1.8020
Moment Y (Twist)	0.2182
Moment Z	16.7459

### 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	6.4336
Shear X	-0.5297
Shear Z	-0.2810
Moment X	-1.0733
Moment Y (Twist)	0.1413
Moment Z	9.4594

# Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States  
 User Name: sales@mtsolar.us  
 Project Name: DOT Region Office - V1Jb  
 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)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
7	6in Pipe Sch 40	6.63	0.28					

ID	Name	d (in)	b (in)	$t_w$ (in)	$t_b$ (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		

ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

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



113	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.05,1.23,1.06,1.06,1.07,1.21,1.06,1.06,1.05,1.05,1.06,1.06,1.05,2.49,1.06,1.06,1.06,1.16	300	200	1
114	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.04,1.06,1.05,1.05,1.06,1.28,1.05,1.05,1.05,1.05,1.05,1.05,3.1,07,1.05,1.05,1.06,1.54	300	200	1
115	19	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.17,1.14,1.14,1.14,1.12,1.09,1.15,1.15,1.15,1.17,1.14,1.14,1.17,1.07,1.14,1.14,1.13,1.10	300	200	1
116	19	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.17,1.17,1.17,1.13,1.17,1.17,1.16,1.10,1.16,1.16,1.16,1.16,1.17,1.17,1.19,1.13,1.17,1.17,1.16,1.09	300	200	1
201	7	25.13	25.13	11.97	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.16,1.18,1.18,1.14,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.15,1.18,1.18,1.16,1.18	300	200	1
204	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.70,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.81,1.68,1.67,1.67,1.65,1.68	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.64,1.67,1.67,1.62,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.64,1.67,1.67,1.65,1.66	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.12,1.18,1.18,1.14,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.13,1.18,1.18,1.16,1.17	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.62,1.67,1.67,1.62,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.62,1.67,1.67,1.65,1.66	300	200	1
208	19	1.33	1.33	2.05	2.08,2.08,2.08,2.08,2.08,2.09,2.09,2.09,1.73,2.09,2.09,2.07,1.45,2.08,2.08,2.08,2.07,2.09,2.09,2.11,1.69,2.09,2.09,2.07,1.36	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.70,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.68,1.68,1.80,1.68,1.67,1.67,1.66,1.68	300	200	1
211	19	1.33	1.33	2.05	2.02,2.02,2.02,2.02,2.02,2.02,1.79,1.79,2.09,1.04,1.74,1.74,1.50,1.40,1.91,1.91,2.06,2.09,1.80,1.80,2.10,1.15,1.72,1.72,1.68,1.44	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.05,1.23,1.06,1.06,1.07,1.21,1.06,1.06,1.05,1.05,1.06,1.06,1.05,2.49,1.06,1.06,1.06,1.16	300	200	1
214	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.04,1.06,1.05,1.05,1.06,1.28,1.05,1.05,1.05,1.05,1.05,1.05,2.1,07,1.05,1.05,1.06,1.54	300	200	1
215	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.15,1.18,1.06,1.15,1.15,1.13,1.12,1.16,1.16,1.17,1.19,1.15,1.15,1.19,1.08,1.15,1.15,1.15,1.12	300	200	1
216	19	6.63	6.63	10.20	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.15,1.19,1.19,1.17,1.13,1.18,1.18,1.18,1.17,1.18,1.18,1.20,1.15,1.19,1.19,1.17,1.12	300	200	1
301	7	25.13	25.13	11.97	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.19,1.17,1.19,1.19,1.14,1.18,1.18,1.18,1.18,1.19,1.19,1.20,1.16,1.19,1.19,1.17,1.18	300	200	1
304	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.71,1.68,1.67,1.67,1.64,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.83,1.69,1.67,1.67,1.65,1.69	300	200	1
305	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.67,1.67,1.62,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.64,1.67,1.67,1.65,1.66	300	200	1
306	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.19,1.12,1.18,1.18,1.09,1.16,1.17,1.17,1.18,1.18,1.18,1.18,1.20,1.12,1.17,1.17,1.15,1.16	300	200	1
307	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.68,1.62,1.67,1.67,1.59,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.61,1.67,1.67,1.64,1.66	300	200	1
308	19	4.20	4.20	2.00	2.33,2.33	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.72,1.68,1.68,1.68,1.63,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.80,1.69,1.67,1.67,1.65,1.69	300	200	1
311	19	4.20	4.20	2.00	2.33,2.33	300	200	1
312	5	1.30	1.30	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.22,1.23,1.22,1.23,1.22,1.22,1.21,1.21,1.28,1.90,1.20,1.20,1.08,1.31,1.22,1.22,1.23,1.24,1.21,1.21,1.29,1.70,1.20,1.20,1.15,1.27	300	200	1
314	19	4.88	4.00	7.50	1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.23,1.17,1.27,1.23,1.23,1.28,1.41,1.23,1.23,1.23,1.24,1.23,1.23,1.0	300	200	1



212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	24.01	6.12	40.24	43.62
214	133.20	85.85	23.49	6.12	40.24	43.62
215	133.20	69.16	16.36	6.12	40.24	43.62
216	133.20	69.16	17.28	6.12	40.24	43.62
301	251.16	69.89	42.30	42.30	75.35	75.35
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	102.39	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	102.39	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	24.71	6.12	40.24	43.62
314	133.20	85.85	24.96	6.12	40.24	43.62
315	133.20	69.16	16.74	6.12	40.24	43.62
316	133.20	69.16	16.73	6.12	40.24	43.62

## 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	0.151	0.396	0.090	0.012	0.006	0.416	#16	0.672	Not Required	Pass
2	0.002	0.314	0.041	0.073	0.007	0.351	#21	0.035	Not Required	Pass
3	0.004	0.516	0.013	0.051	0.004	0.520	#21	0.045	Not Required	Pass
4	0.003	0.497	0.052	0.050	0.013	0.550	#21	0.080	Not Required	Pass
5	0.004	0.320	0.028	0.052	0.006	0.324	#21	0.074	Not Required	Pass
6	0.006	0.691	0.069	0.071	0.019	0.764	#21	0.045	Not Required	Pass
7	0.007	0.428	0.111	0.069	0.028	0.457	#21	0.074	Not Required	Pass
8	0.002	0.143	0.094	0.043	0.010	0.199	#21	0.095	Not Required	Pass
9	0.004	0.077	0.054	0.003	0.003	0.132	#21	0.204	Not Required	Pass
10	0.007	0.648	0.091	0.065	0.019	0.686	#21	0.080	Not Required	Pass
11	0.004	0.141	0.099	0.046	0.010	0.194	#21	0.095	Not Required	Pass
12	0.001	0.487	0.052	0.100	0.009	0.534	#21	0.053	Not Required	Pass
13	0.006	0.114	0.248	0.060	0.013	0.286	#21	0.286	Not Required	Pass
14	0.003	0.105	0.242	0.057	0.013	0.302	#24	0.190	Not Required	Pass
15	0.000	0.020	0.024	0.017	0.003	0.044	#21	Not Required	Not Required	Pass
16	0.000	0.020	0.024	0.016	0.003	0.044	#21	Not Required	Not Required	Pass
101	0.204	0.445	0.011	0.015	0.001	0.468	#16	0.672	Not Required	Pass
102	0.002	0.565	0.064	0.119	0.009	0.630	#21	0.053	Not Required	Pass
103	0.007	0.823	0.043	0.083	0.006	0.870	#21	0.045	Not Required	Pass
104	0.007	0.807	0.113	0.081	0.024	0.882	#21	0.080	Not Required	Pass
105	0.007	0.510	0.121	0.082	0.030	0.542	#21	0.074	Not Required	Pass
106	0.006	0.809	0.037	0.081	0.006	0.843	#21	0.045	Not Required	Pass
107	0.006	0.502	0.104	0.081	0.026	0.531	#21	0.074	Not Required	Pass
108	0.002	0.058	0.091	0.049	0.010	0.149	#21	0.095	Not Required	Pass
109	0.010	0.082	0.034	0.001	0.001	0.121	#21	0.204	Not Required	Pass
110	0.006	0.770	0.103	0.078	0.022	0.852	#21	0.080	Not Required	Pass

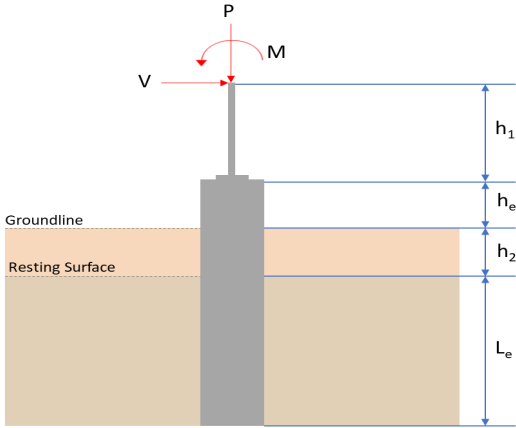
110	0.000	0.779	0.103	0.070	0.022	0.032	#21	0.000	Not Required	Pass
111	0.004	0.058	0.092	0.051	0.010	0.142	#21	0.095	Not Required	Pass
112	0.002	0.544	0.064	0.116	0.010	0.608	#21	0.053	Not Required	Pass
113	0.006	0.256	0.277	0.070	0.014	0.500	#21	0.286	Not Required	Pass
114	0.006	0.274	0.276	0.070	0.014	0.515	#21	0.286	Not Required	Pass
115	0.007	0.372	0.134	0.056	0.011	0.511	#21	0.473	Not Required	Pass
116	0.002	0.354	0.136	0.056	0.011	0.489	#21	0.473	Not Required	Pass
201	0.204	0.445	0.011	0.015	0.001	0.468	#16	0.672	Not Required	Pass
202	0.002	0.544	0.064	0.116	0.010	0.608	#21	0.053	Not Required	Pass
203	0.006	0.809	0.037	0.081	0.006	0.843	#21	0.045	Not Required	Pass
204	0.006	0.779	0.103	0.078	0.022	0.852	#21	0.080	Not Required	Pass
205	0.006	0.502	0.104	0.081	0.026	0.531	#21	0.074	Not Required	Pass
206	0.007	0.823	0.043	0.083	0.006	0.870	#21	0.045	Not Required	Pass
207	0.007	0.510	0.121	0.082	0.030	0.542	#21	0.074	Not Required	Pass
208	0.002	0.078	0.121	0.056	0.011	0.161	#21	0.095	Not Required	Pass
209	0.010	0.082	0.034	0.001	0.001	0.121	#21	0.204	Not Required	Pass
210	0.007	0.807	0.113	0.081	0.024	0.882	#21	0.080	Not Required	Pass
211	0.004	0.092	0.122	0.056	0.011	0.151	#21	0.095	Not Required	Pass
212	0.002	0.565	0.064	0.119	0.009	0.630	#21	0.053	Not Required	Pass
213	0.006	0.256	0.277	0.070	0.014	0.500	#21	0.286	Not Required	Pass
214	0.006	0.274	0.276	0.070	0.014	0.515	#21	0.286	Not Required	Pass
215	0.007	0.259	0.134	0.051	0.010	0.397	#21	0.473	Not Required	Pass
216	0.003	0.227	0.134	0.049	0.010	0.361	#21	0.473	Not Required	Pass
301	0.151	0.396	0.090	0.012	0.006	0.416	#16	0.672	Not Required	Pass
302	0.001	0.487	0.052	0.100	0.009	0.534	#21	0.053	Not Required	Pass
303	0.006	0.691	0.069	0.071	0.019	0.764	#21	0.045	Not Required	Pass
304	0.007	0.648	0.091	0.065	0.019	0.686	#21	0.080	Not Required	Pass
305	0.007	0.428	0.111	0.069	0.028	0.457	#21	0.074	Not Required	Pass
306	0.004	0.516	0.013	0.051	0.004	0.520	#21	0.045	Not Required	Pass
307	0.004	0.320	0.028	0.052	0.006	0.324	#21	0.074	Not Required	Pass
308	0.000	0.020	0.024	0.016	0.003	0.044	#21	Not Required	Not Required	Pass
309	0.004	0.077	0.054	0.003	0.003	0.132	#21	0.204	Not Required	Pass
310	0.003	0.497	0.052	0.050	0.013	0.550	#21	0.080	Not Required	Pass
311	0.000	0.020	0.024	0.017	0.003	0.044	#21	Not Required	Not Required	Pass
312	0.002	0.314	0.041	0.073	0.007	0.351	#21	0.035	Not Required	Pass
313	0.006	0.114	0.248	0.060	0.013	0.286	#21	0.190	Not Required	Pass
314	0.003	0.105	0.242	0.057	0.013	0.302	#24	0.286	Not Required	Pass
315	0.007	0.395	0.133	0.046	0.010	0.529	#21	0.473	Not Required	Pass
316	0.002	0.382	0.134	0.043	0.010	0.516	#21	0.473	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 = 4.75</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resisting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="368 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>6.434</td> <td>10.568</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-0.530</td> <td>-0.895</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.281</td> <td>0.469</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>1.073</td> <td>1.802</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>9.459</td> <td>16.746</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)	6.434	10.568	$V_x$ (kip)	-0.530	-0.895	$V_z$ (kip)	0.281	0.469	$M_x$ (kipft)	1.073	1.802	$M_z$ (kipft)	9.459	16.746	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.53 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.084395 \text{ kip/ft}$																											

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.459 \text{ kipft}) + ((-0.53 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5062 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.5981 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.281 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.044745 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.073 \text{ kipft}) + ((0.281 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17086 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7627 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(4.5981 \text{ ft}), (2.7627 \text{ ft})]$$

$$L_{e,req} = 4.598 \text{ ft}$$

$L_e$  - 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

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

$$Ratio = \frac{(4.598 \text{ ft})}{(4.75 \text{ ft})}$$

$$Ratio = 0.968$$

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

### 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

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

$$q = \frac{(6.434 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40212 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40212 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20106$$

Status: **PASS**  
Ratio: **0.200**

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:**

$H_o = -0.084395 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.5062 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.5062 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.5062 \text{ kipft/ft})) + (4 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2263 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5062 \text{ kipft/ft})) + (3 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.5062 \text{ kipft/ft})) + (2 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20796 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.5062 \text{ kipft/ft})) + ((-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.69448 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2263 \text{ ft})}{2}$$

$$p_a = 0.24197 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20796 \text{ kip/ft}^2)}{(0.24197 \text{ kip/ft}^2)}$$

$$Ratio = 0.85945$$

$p_s$  - 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

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

$$Ratio = \frac{(0.69448 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.97471$$

Status: **PASS**  
Ratio: **0.860**

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

#### Considering z-direction:

$H_o = 0.044745 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.17086 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.17086 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.044745 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.17086 \text{ kipft/ft})) + (4 \times (0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3461 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.17086 \text{ kipft/ft})) + (3 \times (0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 [(3 \times (0.17086 \text{ kipft/ft})) + (2 \times (0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.061869 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.17086 \text{ kipft/ft})) + ((0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.14739 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3461 \text{ ft})}{2}$$

$$p_a = 0.25096 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.061869 \text{ kip/ft}^2)}{(0.25096 \text{ kip/ft}^2)}$$

$$Ratio = 0.24653$$

$p_s$  - 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

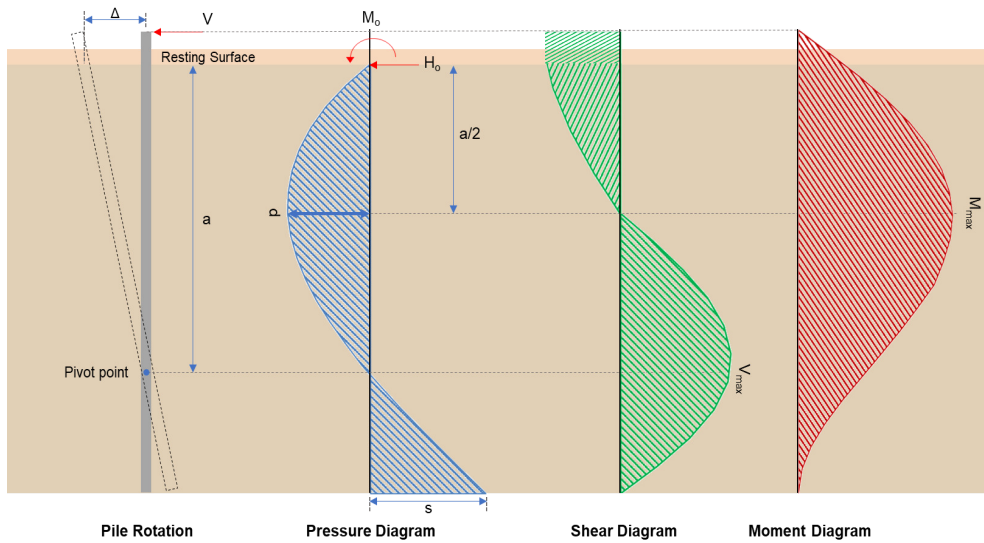
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.14739 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.20687$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.895 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14252 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(16.746 \text{ kipft}) + ((-0.895 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6666 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.6666 \text{ kipft/ft})}{(-0.14252 \text{ kip/ft})}$$

$$E = 18.711 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.6666 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.6666 \text{ kipft/ft})) + (4 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = \frac{(-0.14252 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.6666 \text{ kipft/ft})) + (4 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.224 \text{ ft}$$

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

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

$$V_{max} = ((-0.14252 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.224 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.224 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.3556 \text{ kip}$$

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

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

$$M_{max} = ((-0.14252 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(18.711 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.096 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.469 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.074682 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.802 \text{ kipft}) + ((0.469 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28694 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.28694 \text{ kipft/ft})}{(0.074682 \text{ kip/ft})}$$

$$E = 3.8422 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.28694 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (0.074682 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.28694 \text{ kipft/ft})) + (4 \times (0.074682 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3455 \text{ ft}$$

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

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

$$V_{max} = ((0.074682 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3455 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3455 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.6253 \text{ kip}$$

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

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

$$M_{max} = ((0.074682 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(3.8422 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.3577 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.568 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.245 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.245 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10: Use #3(0.375 in)</p> <p><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p><math>s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p>Ratio - Capacity</p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(10.568 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0039504</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.568 \text{ kip} \rightarrow 10568 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10568 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.89 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.89 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.89 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.89 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.01 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 4.3556 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(4.3556 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.039235$$

**Considering z-direction:**

$V_{max} = 0.6253 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.6253 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.0056327$$

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

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

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 10.096 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(10.096 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.04045$$

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

**Considering z-direction:**

$M_{max} = 1.3577 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(1.3577 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0054393$$

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

REFERENCES	CALCULATIONS	RESULTS
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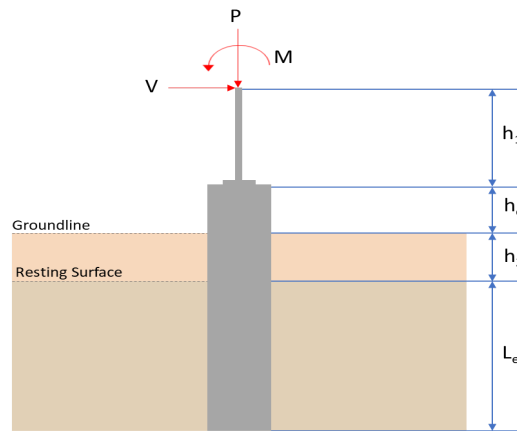
## SkyCiv Foundation Design

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)	6.434	10.568
$V_x$ (kip)	-0.530	-0.895
$V_z$ (kip)	-0.281	-0.469
$M_x$ (kipft)	-1.073	-1.802
$M_z$ (kipft)	9.459	16.746

### 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,

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

$$H_o = \frac{(-0.53 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.084395 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(9.459 \text{ kipft}) + ((-0.53 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5062 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.5981 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.281 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044745 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.073 \text{ kipft}) + ((-0.281 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17086 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0202 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = MAX[(4.5981 \text{ ft}), (2.0202 \text{ ft})]$$

$$L_{e,req} = 4.598 \text{ ft}$$

$L_e$  - 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

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

$$Ratio = \frac{(4.598 \text{ ft})}{(4.75 \text{ ft})}$$

$$Ratio = 0.968$$

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

### 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

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

$$q = \frac{(6.434 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40212 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40212 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20106$$

Status: **PASS**  
Ratio: **0.200**

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:**

$H_o = -0.084395 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.5062 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.5062 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.5062 \text{ kipft/ft})) + (4 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2263 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5062 \text{ kipft/ft})) + (3 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.5062 \text{ kipft/ft})) + (2 \times (-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20796 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.5062 \text{ kipft/ft})) + ((-0.084395 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.69448 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2263 \text{ ft})}{2}$$

$$p_a = 0.24197 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20796 \text{ kip/ft}^2)}{(0.24197 \text{ kip/ft}^2)}$$

$$Ratio = 0.85945$$

$p_s$  - 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

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

$$Ratio = \frac{(0.69448 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.97471$$

Status: **PASS**  
Ratio: **0.860**

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

#### Considering z-direction:

$H_o = -0.044745 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.17086 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.17086 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.044745 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.17086 \text{ kipft/ft})) + (4 \times (-0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3461 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.17086 \text{ kipft/ft})) + (3 \times (-0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (0.17086 \text{ kipft/ft})) + (2 \times (-0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.00079759 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.17086 \text{ kipft/ft})) + ((-0.044745 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.034353 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3461 \text{ ft})}{2}$$

$$p_a = 0.25096 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00079759 \text{ kip/ft}^2)}{(0.25096 \text{ kip/ft}^2)}$$

$$Ratio = 0.0031782$$

$p_s$  - 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

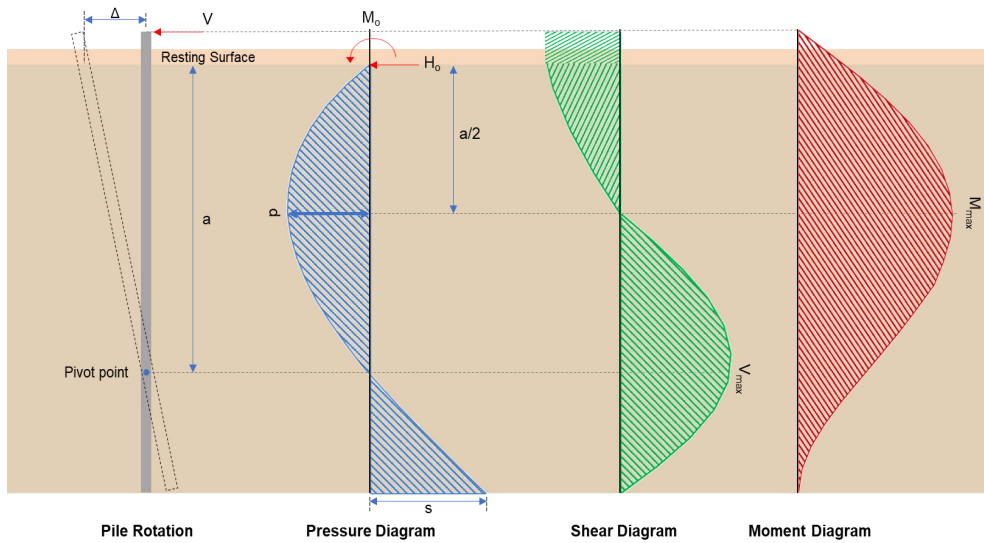
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.034353 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

$$Ratio = 0.048214$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.895 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14252 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(16.746 \text{ kipft}) + ((-0.895 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6666 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.6666 \text{ kipft/ft})}{(-0.14252 \text{ kip/ft})}$$

$$E = 18.711 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.6666 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.6666 \text{ kipft/ft})) + (4 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = \frac{(-0.14252 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (2.6666 \text{ kipft/ft})) + (4 \times (-0.14252 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.224 \text{ ft}$$

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

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

$$V_{max} = ((-0.14252 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.224 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.224 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.3556 \text{ kip}$$

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

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

$$M_{max} = ((-0.14252 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(18.711 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (18.711 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.224 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 10.096 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.469 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.074682 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.802 \text{ kipft}) + ((-0.469 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28694 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.28694 \text{ kipft/ft})}{(-0.074682 \text{ kip/ft})}$$

$$E = 3.8422 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.28694 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.074682 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (0.28694 \text{ kipft/ft})) + (4 \times (-0.074682 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.3455 \text{ ft}$$

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

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

$$V_{max} = ((-0.074682 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3455 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3455 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.6253 \text{ kip}$$

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

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

$$M_{max} = ((-0.074682 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[ \left( \frac{(3.8422 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left( \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8422 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left( \frac{(3.3455 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.3577 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.568 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.245 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.245 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(10.568 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0039504</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48 \text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> <p style="text-align: center;"><math>d = 0.80 D</math></p> <p style="text-align: center;"><math>d = 0.80 \times (48 \text{ in})</math></p> <p style="text-align: center;"><math>d = 38.4 \text{ in}</math></p> <p><math>\lambda_s</math> - size effect modification factor</p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.568 \text{ kip} \rightarrow 10568 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10568 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.89 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.89 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.89 \text{ kip}$$

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

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

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.89 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.01 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 4.3556 \text{ kip}$  - Maximum shear force in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(4.3556 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.039235$$

**Considering z-direction:**

$V_{max} = 0.6253 \text{ kip}$  - Maximum shear force in the z-direction,  
*Ratio* - Capacity

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

$$Ratio = \frac{(0.6253 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.0056327$$

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

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:**

$M_{max} = 10.096 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(10.096 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.04045$$

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

**Considering z-direction:**

$M_{max} = 1.3577 \text{ kipft}$  - Maximum moment in the z-direction,

*Ratio* - Capacity

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

$$Ratio = \frac{(1.3577 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0054393$$

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

REFERENCES	CALCULATIONS	RESULTS
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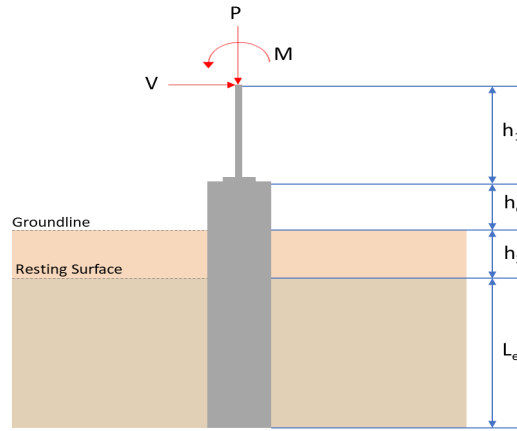
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	8.638	14.259
$V_x$ (kip)	-0.675	-1.132
$V_z$ (kip)	-0.037	-0.060
$M_x$ (kipft)	-0.143	-0.231
$M_z$ (kipft)	10.797	18.815

### 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,

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

$$H_o = \frac{(-0.675 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10748 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(10.797 \text{ kipft}) + ((-0.675 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7193 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7465 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0058917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.143 \text{ kipft}) + ((-0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022771 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.125 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.7465 \text{ ft}), (1.125 \text{ ft})]$$

$$L_{e,req} = 4.746 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.746 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9492$$

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

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

$$q = \frac{(8.638 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53987 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53987 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26994$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.10748 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.7193 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.7193 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.7193 \text{ kipft/ft})) + (4 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4052 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7193 \text{ kipft/ft})) + (3 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.7193 \text{ kipft/ft})) + (2 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.20366 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.7193 \text{ kipft/ft})) + ((-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.69627 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4052 \text{ ft})}{2}$$

$$p_a = 0.25539 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20366 \text{ kip/ft}^2)}{(0.25539 \text{ kip/ft}^2)}$$

$$Ratio = 0.79746$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.69627 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.92836$$

Status: **PASS**  
Ratio: **0.800**

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

**Considering z-direction:**

$H_o = -0.0058917 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.022771 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.022771 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.022771 \text{ kipft/ft})) + (4 \times (-0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5263 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.022771 \text{ kipft/ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 [(3 \times (0.022771 \text{ kipft/ft})) + (2 \times (-0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.0000234 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 [(2 \times (0.022771 \text{ kipft/ft})) + ((-0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.0038599 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5263 \text{ ft})}{2}$$

$$p_a = 0.26447 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0000234 \text{ kip/ft}^2)}{(0.26447 \text{ kip/ft}^2)}$$

$$Ratio = 0.000088477$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

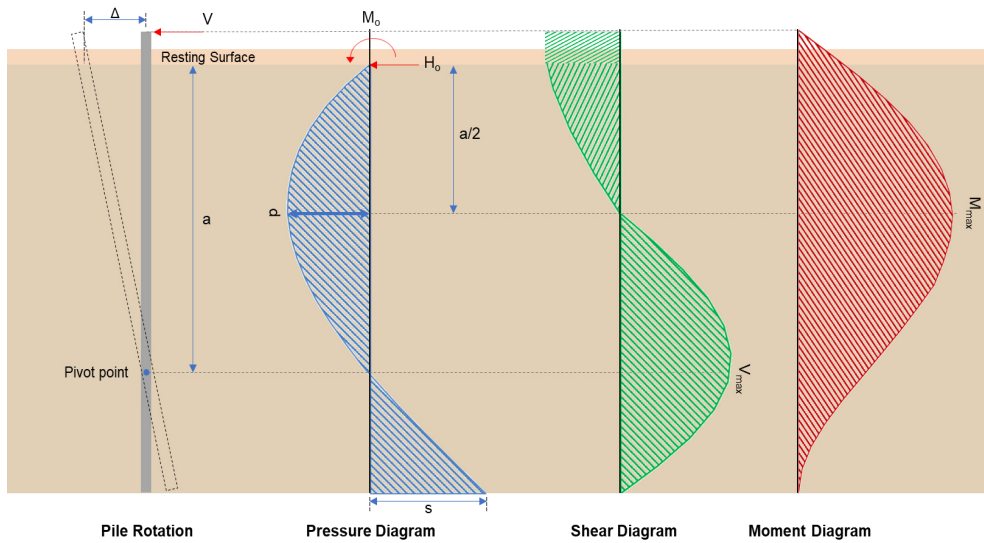
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0038599 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.0051465$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.132 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.815 \text{ kipft}) + ((-1.132 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.996 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.996 \text{ kipft/ft})}{(-0.18025 \text{ kip/ft})}$$

$$E = 16.621 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.996 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.996 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(6 \times (2.996 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft}))}{}$$

$$a = 3.4029 \text{ ft}$$

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

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

$$V_{max} = ((-0.18025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4029 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4029 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7217 \text{ kip}$$

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

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

$$M_{max} = ((-0.18025 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(16.621 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.474 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.06 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0095541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.231 \text{ kipft}) + ((-0.06 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036783 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.036783 \text{ kipft/ft})}{(-0.0095541 \text{ kip/ft})}$$

$$E = 3.85 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.036783 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.0095541 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.036783 \text{ kipft/ft})) + (4 \times (-0.0095541 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5267 \text{ ft}$$

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

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

$$V_{max} = ((-0.0095541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.85 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.5267 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.85 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.5267 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.077381 \text{ kip}$$

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

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

$$M_{max} = ((-0.0095541 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(3.85 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5267 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.85 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.5267 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.85 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.5267 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1763 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(14.259 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.122 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.122 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(14.259 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0053301</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 14.259 \text{ kip} \rightarrow 14259 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(14259 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.39 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.39 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.39 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.33 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 4.7217 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.7217 \text{ kip})}{(111.33 \text{ kip})}$$

$$Ratio = 0.042411$$

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

**Considering z-direction:**

$V_{max} = 0.077381 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.077381 \text{ kip})}{(111.33 \text{ kip})}$$

$$Ratio = 0.00069504$$

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

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

$S_m$  - Section modulus

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

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

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

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

Allowable flexural strength:

$M_n$  shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 11.474 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(11.474 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.045972$$

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

**Considering z-direction:**

$M_{max} = 0.1763 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.1763 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00070634$$

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

REFERENCES	CALCULATIONS	RESULTS
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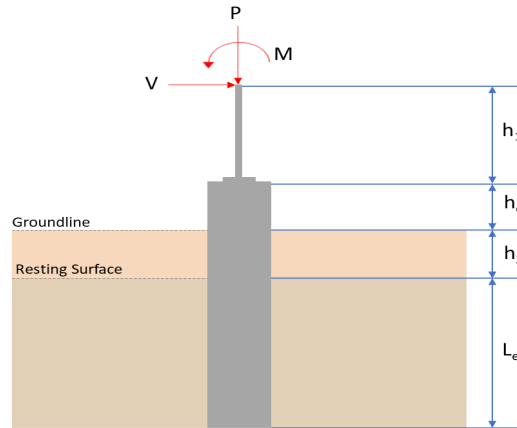
## SkyCiv Foundation Design

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

$L = 5$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

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

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	8.638	14.259
$V_x$ (kip)	-0.675	-1.132
$V_z$ (kip)	0.037	0.060
$M_x$ (kipft)	0.143	0.230
$M_z$ (kipft)	10.797	18.815

### 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,

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

$$H_o = \frac{(-0.675 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10748 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(10.797 \text{ kipft}) + ((-0.675 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7193 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7465 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0058917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.143 \text{ kipft}) + ((0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.022771 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3176 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.7465 \text{ ft}), (1.3176 \text{ ft})]$$

$$L_{e,req} = 4.746 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.746 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9492$$

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

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

$$q = \frac{(8.638 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53987 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53987 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26994$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.10748 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 1.7193 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (1.7193 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.7193 \text{ kipft/ft})) + (4 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4052 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7193 \text{ kipft/ft})) + (3 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.7193 \text{ kipft/ft})) + (2 \times (-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.20366 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (1.7193 \text{ kipft/ft})) + ((-0.10748 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.69627 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4052 \text{ ft})}{2}$$

$$p_a = 0.25539 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20366 \text{ kip/ft}^2)}{(0.25539 \text{ kip/ft}^2)}$$

$$Ratio = 0.79746$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.69627 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.92836$$

Status: **PASS**  
Ratio: **0.800**

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

#### Considering z-direction:

$H_o = 0.0058917 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.022771 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.022771 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.022771 \text{ kipft/ft})) + (4 \times (0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5263 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.022771 \text{ kipft/ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.022771 \text{ kipft/ft})) + (2 \times (0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.0075939 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.022771 \text{ kipft/ft})) + ((0.0058917 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.018 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5263 \text{ ft})}{2}$$

$$p_a = 0.26447 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0075939 \text{ kip/ft}^2)}{(0.26447 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.028713$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

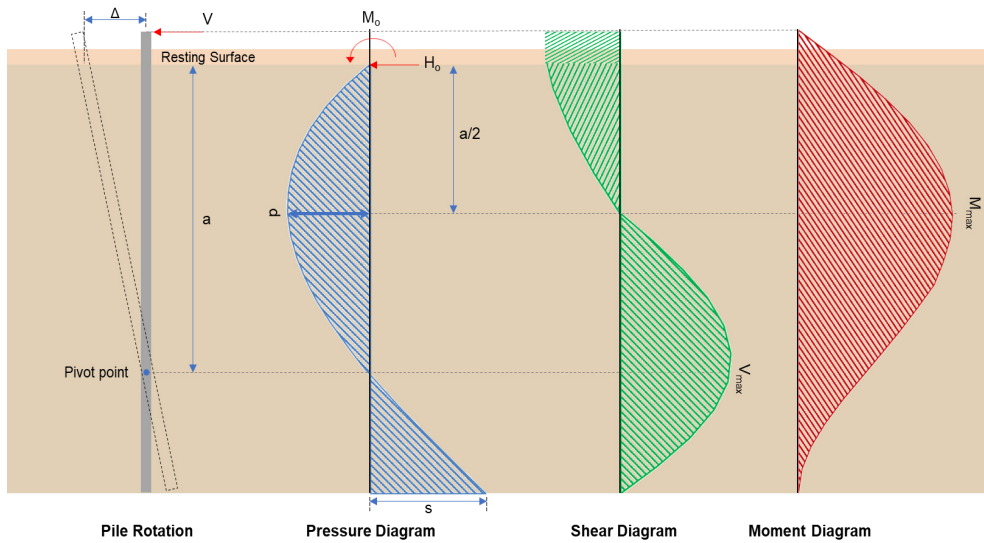
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.018 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.024$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.132 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18025 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(18.815 \text{ kipft}) + ((-1.132 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.996 \text{ kipft/ft}$$

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

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

$$E = \frac{(2.996 \text{ kipft/ft})}{(-0.18025 \text{ kip/ft})}$$

$$E = 16.621 \text{ ft}$$

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

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

$$a = \frac{(4 \times (2.996 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.996 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = \frac{(6 \times (2.996 \text{ kipft/ft})) + (4 \times (-0.18025 \text{ kip/ft}) \times (5 \text{ ft}))}{}$$

$$a = 3.4029 \text{ ft}$$

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

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

$$V_{max} = ((-0.18025 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4029 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4029 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7217 \text{ kip}$$

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

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

$$M_{max} = ((-0.18025 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(16.621 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (16.621 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4029 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.474 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.06 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0095541 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.23 \text{ kipft}) + ((0.06 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.036624 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.036624 \text{ kipft/ft})}{(0.0095541 \text{ kip/ft})}$$

$$E = 3.8333 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.036624 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.0095541 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.036624 \text{ kipft/ft})) + (4 \times (0.0095541 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5271 \text{ ft}$$

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

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

$$V_{max} = ((0.0095541 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.8333 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.5271 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.8333 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.5271 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.077157 \text{ kip}$$

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

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

$$M_{max} = ((0.0095541 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(3.8333 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5271 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.8333 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.5271 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.8333 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.5271 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.17574 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(14.259 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.122 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.122 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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. 10: 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>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(14.259 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0053301$	<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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 14.259 \text{ kip} \rightarrow 14259 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(14259 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.39 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.39 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.39 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.39 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.33 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 4.7217 \text{ kip}$  - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(4.7217 \text{ kip})}{(111.33 \text{ kip})}$$

$$Ratio = 0.042411$$

**Considering z-direction:**

$V_{max} = 0.077157 \text{ kip}$  - Maximum shear force in the z-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(0.077157 \text{ kip})}{(111.33 \text{ kip})}$$

$$Ratio = 0.00069303$$

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

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:**

$M_{max} = 11.474 \text{ kipft}$  - Maximum moment in the x-direction,

$Ratio$  - Capacity

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

$$Ratio = \frac{(11.474 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.045972$$

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

**Considering z-direction:**

$M_{max} = 0.17574 \text{ kipft}$  - Maximum moment in the z-direction,

$Ratio$  - Capacity

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

$$\text{Ratio} = \frac{(0.17574 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0007041$$

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