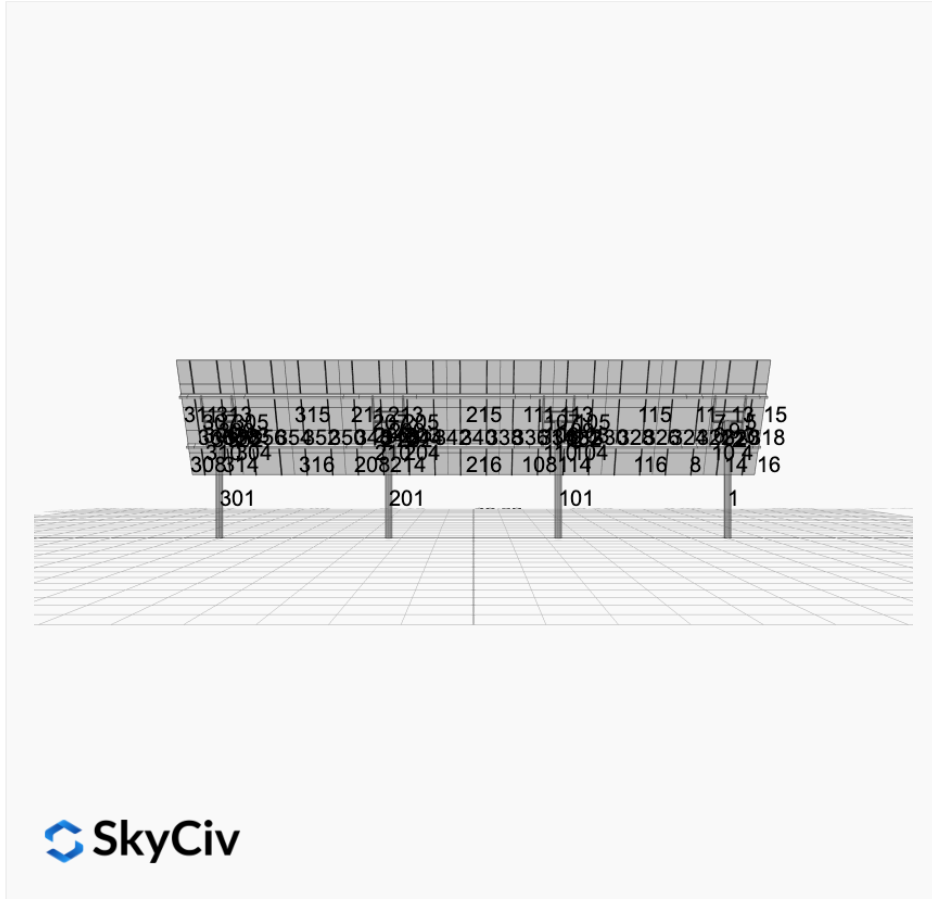


# Project Details



**Project Name:** MTSOLAR\_7FB990A0BCGL - V2Jb      **Date:** Mon Mar 03 2025  
**Location:** 106 Allen Farm Rd, Northwood, NH 03261, USA      **Number of Modules:** 55  
**Unique ID:** 4P-22.5-10TOP-XD-12-L-5Hx11W-EH55      **Number of Poles:** 4  
**Dealer:** \_\_\_\_\_      **Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	17.29 ft
<b>Array Dimensions E/W</b>	77.00 ft
<b>Winter Tilt Angle</b>	60
<b>Front Edge Clearance</b>	8 ft

## MT Solar Bill of Materials (4P-22.5-10TOP-XD-12-L-5Hx11W-EH55)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	12
MTS-CLAMP-ANGLE-4PK	Angle Clamp	11

## Rail Bill of Materials

Part	Qty
Rails (205in)	22
Rail Attachment	88
Module Mid Clamp	88

<b>Part</b>	<b>Qty</b>
Module End Clamp	44
Ground Lug	11

## Site Details:



**Site Address:** 106 Allen Farm Rd, Northwood, NH 03261, USA

### Array Specification

<b>Duty Classification:</b>	XD
<b>Module Width:</b>	41.00 in
<b>Module Length:</b>	83.00in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	11
<b>Total Number of Modules:</b>	55
<b>Winter Tilt Angle:</b>	60
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	22.98 ft
<b>Total Frame Length:</b>	77.00 ft
<b>Frame Weight:</b>	7998 lbs
<b>Array Dimensions N/S:</b>	17.29 ft
<b>Array Dimensions E/W:</b>	77.00 ft
<b>Rail Length:</b>	207.50 in
<b>Rail Spacing:</b>	3.50 ft

### Support Specifications

<b>Pole Size:</b>	10in Pipe Sch 80
<b>Pole Length above Grade:</b>	15.49 ft
<b>Number of Poles:</b>	4
<b>Pole Spacing:</b>	22.5 ft

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 8.75 ft Pile 2: 9.75 ft Pile 3: 9.75 ft Pile 4: 8.75 ft
<b>Foundation Volume:</b>	21.926 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	106 Allen Farm Rd, Northwood, NH 03261, USA
<b>Wind Speed:</b>	135 mph

**Snow Load:**

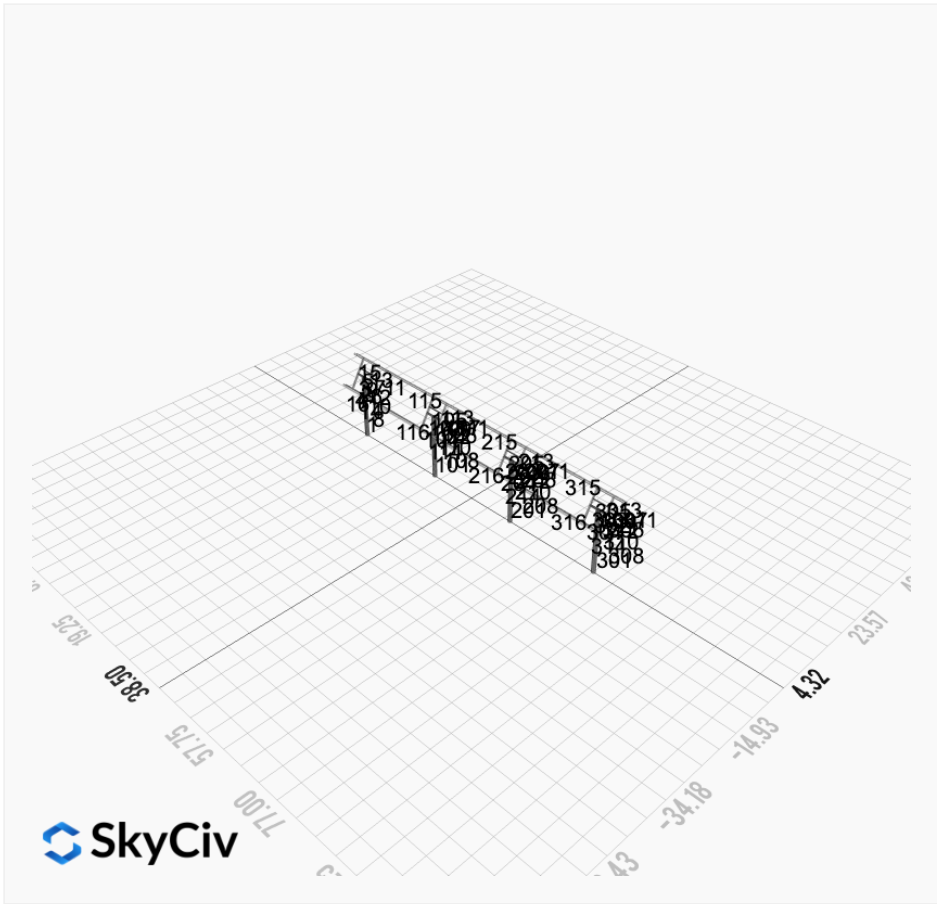
60 psf

### **Design Disclaimer**

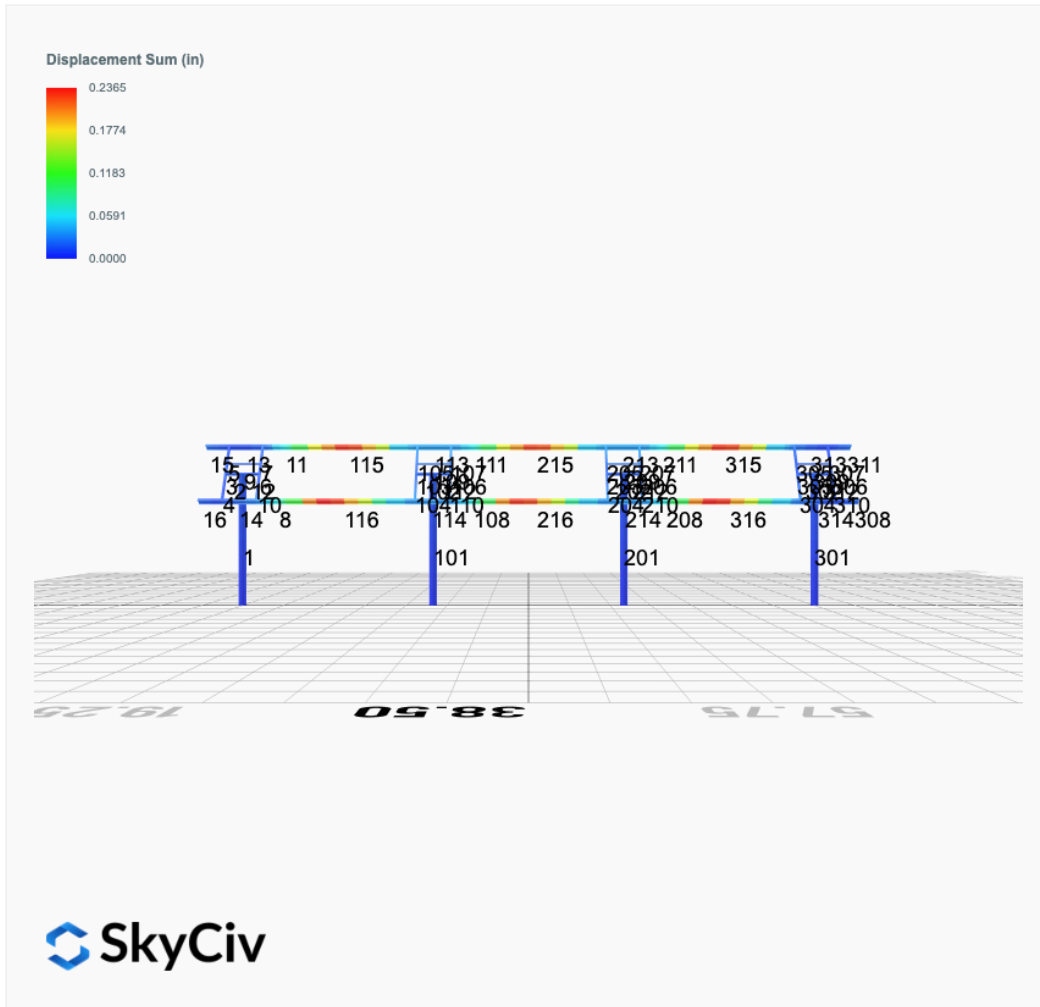
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.



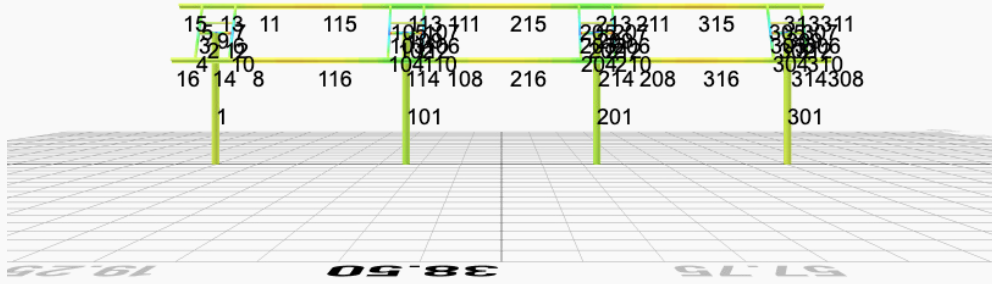




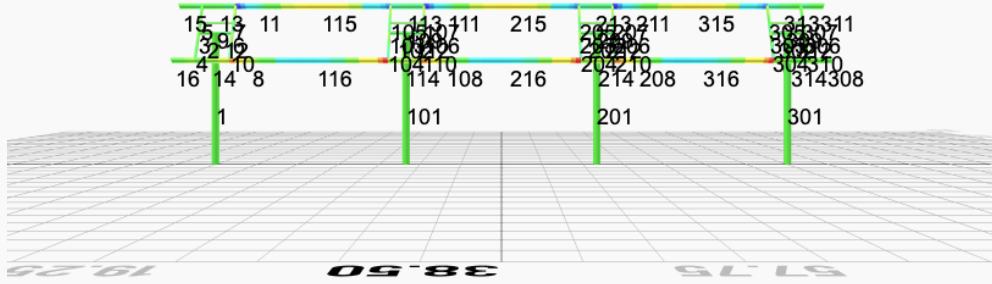
# FEM Results (Envelope Worst Case for each member)



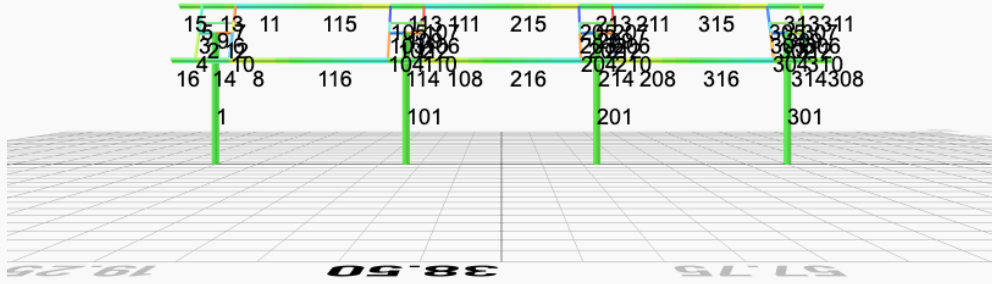
Top Bending Stress Z (ksi)



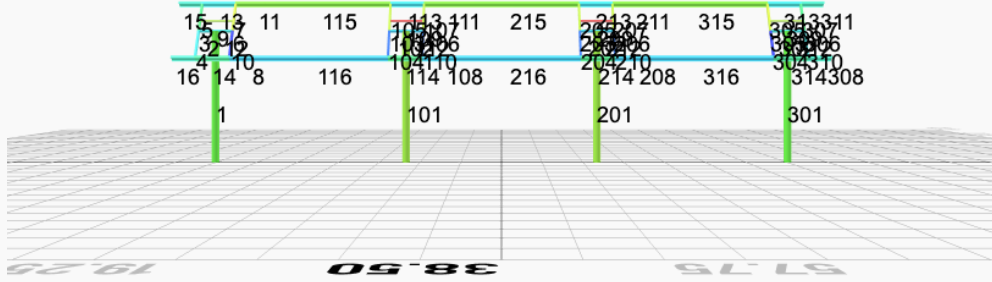
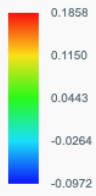
Top Bending Stress Y (ksi)



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	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 2. D + L	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 3. D + (S or Lr or R)	0.0632	3.7705	0.1500	0.6898	-0.2516	-0.9368
ULS: 3. D + (S or Lr or R)	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0575	3.5459	0.1365	0.6279	-0.2290	-0.8512
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 5b. D + 0.7E	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0575	3.5459	0.1365	0.6279	-0.2290	-0.8512
ULS: 8. 0.6D + 0.7E	0.0243	1.7232	0.0577	0.2652	-0.0967	-0.3566
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.4336	5.4141	0.4238	1.8180	-4.3242	69.5990
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.5086	0.3337	-0.2262	-0.9106	3.9394	-69.7103
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2981	5.4525	0.3823	1.6598	-3.3513	51.7939
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0575	3.5459	0.1365	0.6279	-0.2290	-0.8512
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4086	1.6422	-0.1052	-0.3866	2.8464	-52.6882
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0575	3.5459	0.1365	0.6279	-0.2290	-0.8512
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3151	4.7786	0.3419	1.4740	-3.2834	52.0507
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3916	0.9682	-0.1456	-0.5724	2.9143	-52.4313
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0405	2.8720	0.0961	0.4420	-0.1611	-0.5943
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.4498	4.2654	0.3854	1.6411	-4.2597	69.8368
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0243	1.7232	0.0577	0.2652	-0.0967	-0.3566
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4924	-0.8151	-0.2646	-1.0874	4.0038	-69.4726
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0243	1.7232	0.0577	0.2652	-0.0967	-0.3566

### 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	8.1342
Shear X	-7.5100
Shear Z	0.6915
Moment X	2.9607
Moment Y (Twist)	7.2087
Moment Z	117.0231

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.4525
Shear X	-4.5086
Shear Z	0.4238
Moment X	1.8180
Moment Y (Twist)	4.3242
Moment Z	69.8368

## 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	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 2. D + L	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 3. D + (S or Lr or R)	-0.0632	4.8818	-0.0021	-0.0126	0.0413	0.9770
ULS: 3. D + (S or Lr or R)	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0575	4.5574	-0.0019	-0.0115	0.0376	0.8907

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 5b. D + 0.7E	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0575	4.5574	-0.0019	-0.0115	0.0376	0.8907
ULS: 8. 0.6D + 0.7E	-0.0243	2.1505	-0.0008	-0.0049	0.0159	0.3792
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.5980	7.4111	0.0414	0.1655	-0.6120	102.8333
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.5230	-0.2466	-0.0411	-0.1694	0.6272	-99.5953
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.9757	7.4276	0.0301	0.1187	-0.4412	77.5417
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0575	4.5574	-0.0019	-0.0115	0.0376	0.8907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8651	1.6843	-0.0317	-0.1324	0.4881	-74.2797
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0575	4.5574	-0.0019	-0.0115	0.0376	0.8907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.9586	6.4544	0.0307	0.1221	-0.4524	77.2830
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8821	0.7111	-0.0312	-0.1291	0.4770	-74.5385
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0405	3.5842	-0.0014	-0.0081	0.0264	0.6320
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.5818	5.9774	0.0419	0.1687	-0.6225	102.5805
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0243	2.1505	-0.0008	-0.0049	0.0159	0.3792
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.5392	-1.6803	-0.0405	-0.1661	0.6166	-99.8481
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0243	2.1505	-0.0008	-0.0049	0.0159	0.3792

### 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	11.3265
Shear X	-10.9856
Shear Z	0.0712
Moment X	0.2893
Moment Y (Twist)	1.0640
Moment Z	172.7126

### 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	7.4276
Shear X	-6.5980
Shear Z	0.0419
Moment X	-0.1694
Moment Y (Twist)	0.6272
Moment Z	102.8333

### 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	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 2. D + L	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 3. D + (S or Lr or R)	-0.0632	4.8818	0.0021	0.0126	-0.0411	0.9770
ULS: 3. D + (S or Lr or R)	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0575	4.5574	0.0019	0.0114	-0.0374	0.8907
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 5b. D + 0.7E	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0575	4.5574	0.0019	0.0114	-0.0374	0.8907
ULS: 8. 0.6D + 0.7E	-0.0243	2.1505	0.0008	0.0048	-0.0158	0.3792
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.5980	7.4111	-0.0414	-0.1655	0.6120	102.8333
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.5230	-0.2466	0.0411	0.1693	-0.6271	-99.5953
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320

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	-4.9757	7.4276	-0.0301	-0.1187	0.4413	77.5417
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0575	4.5574	0.0019	0.0114	-0.0374	0.8907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8651	1.6843	0.0317	0.1324	-0.4880	-74.2797
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0575	4.5574	0.0019	0.0114	-0.0374	0.8907
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.9586	6.4544	-0.0307	-0.1221	0.4525	77.2830
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8821	0.7111	0.0312	0.1290	-0.4769	-74.5385
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0405	3.5842	0.0014	0.0081	-0.0263	0.6320
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.5818	5.9774	-0.0419	-0.1687	0.6226	102.5805
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0243	2.1505	0.0008	0.0048	-0.0158	0.3792
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.5392	-1.6803	0.0405	0.1661	-0.6166	-99.8481
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0243	2.1505	0.0008	0.0048	-0.0158	0.3792

### 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	11.3265
Shear X	-10.9856
Shear Z	-0.0712
Moment X	-0.2893
Moment Y (Twist)	1.0640
Moment Z	172.7130

### 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	7.4276
Shear X	-6.5980
Shear Z	-0.0419
Moment X	0.1693
Moment Y (Twist)	0.6271
Moment Z	102.8333

### 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	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 2. D + L	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 3. D + (S or Lr or R)	0.0632	3.7705	-0.1500	-0.6902	0.2517	-0.9367
ULS: 3. D + (S or Lr or R)	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0575	3.5459	-0.1365	-0.6282	0.2291	-0.8511
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 5b. D + 0.7E	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0575	3.5459	-0.1365	-0.6282	0.2291	-0.8511
ULS: 8. 0.6D + 0.7E	0.0243	1.7232	-0.0577	-0.2654	0.0967	-0.3566
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.4336	5.4141	-0.4238	-1.8182	4.3242	69.5991
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.5086	0.3337	0.2262	0.9103	-3.9394	-69.7102
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2981	5.4525	-0.3823	-1.6601	3.3514	51.7939
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0575	3.5459	-0.1365	-0.6282	0.2291	-0.8511
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4086	1.6422	0.1052	0.3863	-2.8463	-52.6881
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0575	3.5459	-0.1365	-0.6282	0.2291	-0.8511
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.3151	4.7786	-0.3419	-1.4742	3.2835	52.0508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.3916	0.9682	0.1456	0.5722	-2.9142	-52.4312
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0405	2.8719	-0.0961	-0.4423	0.1612	-0.5943

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.4498	4.2654	-0.3854	-1.6413	4.2598	69.8368
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0243	1.7232	-0.0577	-0.2654	0.0967	-0.3566
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.4924	-0.8151	0.2646	1.0872	-4.0038	-69.4725
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0243	1.7232	-0.0577	-0.2654	0.0967	-0.3566

### 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	8.1341
Shear X	-7.5100
Shear Z	-0.6915
Moment X	-2.9629
Moment Y (Twist)	7.2098
Moment Z	117.0248

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.4525
Shear X	-4.5086
Shear Z	-0.4238
Moment X	-1.8182
Moment Y (Twist)	4.3242
Moment Z	69.8368

# Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States  
 User Name: sales@mtsolar.us  
 Project Name: MTSOLAR\_7FB990A0BCGL - V2Jb  
 Unit System: imperial



## Design Input Information

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

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

**Section Dimensions**

ID	Name	d (in)	$t_w$ (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
12	10in Pipe Sch 80	10.75	0.59					

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

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

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







212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	32.07	6.46	56.26	44.91
214	159.30	97.43	31.98	6.46	56.26	44.91
215	159.30	48.27	14.68	6.46	56.26	44.91
216	159.30	48.27	15.20	6.46	56.26	44.91
301	851.50	360.06	229.67	229.67	255.45	255.45
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	137.23	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	137.23	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	36.48	6.46	56.26	44.91
314	159.30	97.43	36.05	6.46	56.26	44.91
315	159.30	48.27	14.48	6.46	56.26	44.91
316	159.30	48.27	14.46	6.46	56.26	44.91

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.023	0.510	0.034	0.029	0.003	0.533	#13	0.542	Not Required	Pass
2	0.002	0.072	0.113	0.024	0.031	0.172	#13	0.054	Not Required	Pass
3	0.003	0.281	0.025	0.025	0.008	0.284	#13	0.046	Not Required	Pass
4	0.003	0.285	0.062	0.029	0.014	0.348	#13	0.122	Not Required	Pass
5	0.002	0.175	0.024	0.028	0.008	0.179	#13	0.076	Not Required	Pass
6	0.008	0.650	0.073	0.067	0.020	0.709	#13	0.046	Not Required	Pass
7	0.008	0.403	0.148	0.064	0.036	0.423	#13	0.076	Not Required	Pass
8	0.006	0.129	0.199	0.037	0.016	0.216	#23	0.102	Not Required	Pass
9	0.007	0.076	0.106	0.006	0.006	0.156	#13	0.206	Not Required	Pass
10	0.009	0.607	0.151	0.060	0.033	0.618	#13	0.082	Not Required	Pass
11	0.006	0.114	0.204	0.041	0.016	0.211	#23	0.102	Not Required	Pass
12	0.002	0.343	0.378	0.068	0.071	0.722	#13	0.054	Not Required	Pass
13	0.008	0.086	0.418	0.050	0.019	0.450	#23	0.306	Not Required	Pass
14	0.006	0.064	0.412	0.047	0.019	0.429	#23	0.204	Not Required	Pass
15	0.000	0.003	0.007	0.006	0.002	0.009	#21	Not Required	Not Required	Pass
16	0.000	0.003	0.007	0.006	0.002	0.009	#21	Not Required	Not Required	Pass
101	0.031	0.752	0.004	0.043	0.000	0.769	#13	0.542	Not Required	Pass
102	0.004	0.280	0.352	0.066	0.070	0.634	#13	0.036	Not Required	Pass
103	0.007	0.640	0.038	0.063	0.001	0.671	#13	0.046	Not Required	Pass
104	0.007	0.670	0.154	0.067	0.033	0.750	#13	0.082	Not Required	Pass
105	0.007	0.397	0.159	0.063	0.041	0.428	#13	0.076	Not Required	Pass
106	0.007	0.698	0.039	0.070	0.004	0.724	#13	0.046	Not Required	Pass
107	0.007	0.434	0.151	0.069	0.040	0.465	#13	0.076	Not Required	Pass
108	0.007	0.052	0.184	0.042	0.015	0.218	#21	0.102	Not Required	Pass
109	0.018	0.039	0.069	0.002	0.001	0.114	#13	0.206	Not Required	Pass
110	0.007	0.698	0.143	0.060	0.021	0.750	#13	0.082	Not Required	Pass

110	0.007	0.096	0.143	0.009	0.051	0.739	#13	0.062	Not Required	Pass
111	0.006	0.067	0.190	0.042	0.015	0.210	#21	0.102	Not Required	Pass
112	0.004	0.310	0.390	0.069	0.076	0.702	#13	0.036	Not Required	Pass
113	0.009	0.182	0.424	0.053	0.019	0.516	#21	0.306	Not Required	Pass
114	0.010	0.241	0.419	0.056	0.019	0.544	#21	0.306	Not Required	Pass
115	0.017	0.656	0.218	0.043	0.016	0.814	#13	0.644	Not Required	Pass
116	0.008	0.619	0.215	0.046	0.016	0.777	#13	0.644	Not Required	Pass
201	0.031	0.752	0.004	0.043	0.000	0.769	#13	0.542	Not Required	Pass
202	0.004	0.310	0.390	0.069	0.076	0.702	#13	0.036	Not Required	Pass
203	0.007	0.698	0.039	0.070	0.004	0.724	#13	0.046	Not Required	Pass
204	0.007	0.698	0.143	0.069	0.031	0.759	#13	0.082	Not Required	Pass
205	0.007	0.434	0.151	0.069	0.040	0.465	#13	0.076	Not Required	Pass
206	0.007	0.640	0.038	0.063	0.001	0.671	#13	0.046	Not Required	Pass
207	0.007	0.397	0.159	0.063	0.041	0.428	#13	0.076	Not Required	Pass
208	0.006	0.070	0.205	0.046	0.016	0.226	#21	0.102	Not Required	Pass
209	0.018	0.039	0.069	0.002	0.001	0.114	#13	0.206	Not Required	Pass
210	0.007	0.670	0.154	0.067	0.033	0.750	#13	0.082	Not Required	Pass
211	0.006	0.100	0.209	0.043	0.016	0.212	#21	0.102	Not Required	Pass
212	0.004	0.280	0.352	0.066	0.070	0.634	#13	0.036	Not Required	Pass
213	0.009	0.182	0.425	0.053	0.019	0.516	#21	0.306	Not Required	Pass
214	0.010	0.241	0.419	0.056	0.019	0.544	#21	0.306	Not Required	Pass
215	0.017	0.517	0.218	0.042	0.015	0.673	#13	0.644	Not Required	Pass
216	0.011	0.411	0.214	0.042	0.015	0.568	#13	0.644	Not Required	Pass
301	0.023	0.510	0.034	0.029	0.003	0.533	#13	0.542	Not Required	Pass
302	0.002	0.343	0.378	0.068	0.071	0.722	#13	0.054	Not Required	Pass
303	0.008	0.650	0.073	0.067	0.020	0.709	#13	0.046	Not Required	Pass
304	0.009	0.607	0.151	0.060	0.033	0.618	#13	0.082	Not Required	Pass
305	0.008	0.403	0.148	0.064	0.036	0.423	#13	0.076	Not Required	Pass
306	0.003	0.281	0.025	0.025	0.008	0.284	#13	0.046	Not Required	Pass
307	0.002	0.175	0.024	0.028	0.008	0.179	#13	0.076	Not Required	Pass
308	0.000	0.003	0.007	0.006	0.002	0.009	#21	Not Required	Not Required	Pass
309	0.007	0.076	0.106	0.006	0.006	0.156	#13	0.206	Not Required	Pass
310	0.003	0.285	0.062	0.029	0.014	0.348	#13	0.122	Not Required	Pass
311	0.000	0.003	0.007	0.006	0.002	0.009	#21	Not Required	Not Required	Pass
312	0.002	0.072	0.113	0.024	0.031	0.172	#13	0.054	Not Required	Pass
313	0.008	0.086	0.418	0.050	0.019	0.450	#23	0.204	Not Required	Pass
314	0.006	0.064	0.412	0.047	0.019	0.429	#23	0.306	Not Required	Pass
315	0.017	0.662	0.219	0.041	0.016	0.818	#13	0.644	Not Required	Pass
316	0.008	0.650	0.212	0.037	0.016	0.807	#13	0.644	Not Required	Pass

## Definitions

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

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



REFERENCES	CALCULATIONS	RESULTS
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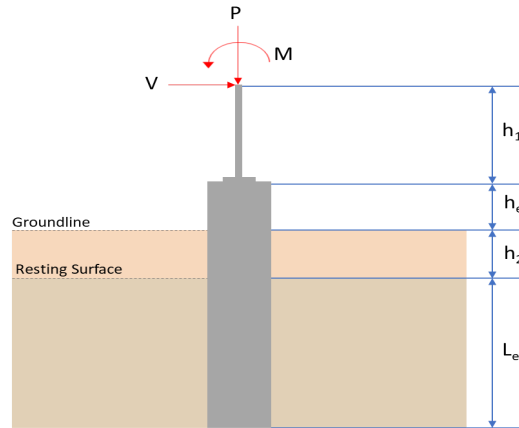
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: rectangular

$b = 48$  in - Pile width

$D = 48$  in - Pile depth

$L = 8.75$  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)	5.453	8.134
$V_x$ (kip)	-4.509	-7.510
$V_z$ (kip)	0.424	0.691
$M_x$ (kipft)	1.818	2.961
$M_z$ (kipft)	69.837	117.023

### 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,

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

$$H_o = \frac{(-4.509 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.71799 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(69.837 \text{ kipft}) + ((-4.509 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.121 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.1384 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.424 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.067516 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.818 \text{ kipft}) + ((0.424 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28949 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.3205 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(8.1384 \text{ ft}), (3.3205 \text{ ft})]$$

$$L_{e,req} = 8.138 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(8.138 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.93006$$

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

**End-bearing Capacity (ASD)**

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.453 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.34081 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17041$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.71799 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 11.121 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (11.121 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.121 \text{ kipft/ft})) + (4 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0328 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.121 \text{ kipft/ft})) + (3 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (11.121 \text{ kipft/ft})) + (2 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.30954 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (11.121 \text{ kipft/ft})) + ((-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2506 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0328 \text{ ft})}{2}$$

$$p_a = 0.45246 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30954 \text{ kip/ft}^2)}{(0.45246 \text{ kip/ft}^2)}$$

$$Ratio = 0.68411$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2506 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.95287$$

Status: **PASS**  
Ratio: **0.680**

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

#### Considering z-direction:

$H_o = 0.067516 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.28949 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.28949 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.067516 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.28949 \text{ kipft/ft})) + (4 \times (0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2536 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.28949 \text{ kipft/ft})) + (3 \times (0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 [(3 \times (0.28949 \text{ kipft/ft})) + (2 \times (0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.04103 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.28949 \text{ kipft/ft})) + ((0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 0.09167 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2536 \text{ ft})}{2}$$

$$p_a = 0.46902 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.04103 \text{ kip/ft}^2)}{(0.46902 \text{ kip/ft}^2)}$$

$$Ratio = 0.08748$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

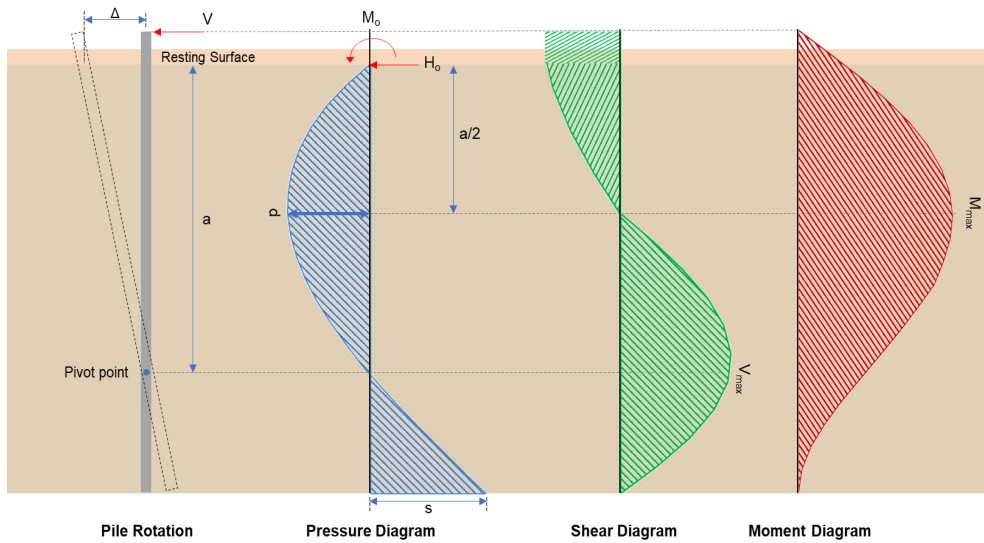
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.09167 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.069844$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-7.51 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1959 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(117.02 \text{ kipft}) + ((-7.51 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 18.634 \text{ kipft/ft}$$

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

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

$$E = \frac{(18.634 \text{ kipft/ft})}{(-1.1959 \text{ kip/ft})}$$

$$E = 15.582 \text{ ft}$$

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

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

$$a = \frac{(4 \times (18.634 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.1959 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times 18.634 \text{ kipft/ft}) + (4 \times (-1.1959 \text{ kip/ft}) \times 8.75 \text{ ft})}$$

$$a = \frac{(6 \times (18.634 \text{ kipft/ft})) + (4 \times (-1.1959 \text{ kip/ft}) \times (8.75 \text{ ft}))}{}$$

$$a = 6.0319 \text{ ft}$$

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

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

$$V_{max} = ((-1.1959 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.582 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.0319 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.582 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.0319 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 18.229 \text{ kip}$$

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

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

$$M_{max} = ((-1.1959 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[ \left( \frac{(15.582 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.582 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.582 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 75.95 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.691 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.11003 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(2.961 \text{ kipft}) + ((0.691 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.4715 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.4715 \text{ kipft/ft})}{(0.11003 \text{ kip/ft})}$$

$$E = 4.2851 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.4715 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.11003 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.4715 \text{ kipft/ft})) + (4 \times (0.11003 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2537 \text{ ft}$$

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

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

$$V_{max} = ((0.11003 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.2851 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.2537 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (4.2851 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.2537 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.67474 \text{ kip}$$

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

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

$$M_{max} = ((0.11003 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[ \left( \frac{(4.2851 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.2537 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (4.2851 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.2537 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.2851 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.2537 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.6086 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.134 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.326 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.326 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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 <math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> <p>25.7.2.2 Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p>25.7.2.1 <math>s_{ties}</math> - Maximum spacing of ties,</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 d_{bar})]</math></p> <p style="text-align: center;"><math>s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p style="text-align: center;"><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p style="text-align: center;"><math>s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]</math></p> <p style="text-align: center;"><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 style="text-align: center;"><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>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 <math>\phi P_N</math> - Allowable axial compressive strength</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]</math></p> <p style="text-align: center;"><math>\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]</math></p> <p style="text-align: center;"><math>\phi P_N = 2675.2 \text{ kip}</math></p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;"><math>Ratio = \frac{P}{\phi P_N}</math></p> <p style="text-align: center;"><math>Ratio = \frac{(8.134 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0030405</math></p>	<p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
<p>22.5.2.2 <math>b_w</math> = 48 in - Effective width, <math>d</math> - Effective depth</p> <p>22.5.5.1.3 <math>\lambda_s</math> - size effect modification factor</p> <p>22.5.5.1.1 <math>V_{c,max}</math> - Max shear strength of concrete</p>	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></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 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 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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.134 \text{ kip} \rightarrow 8134 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8134 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 18.229 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.229 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.16452$$

Status: **PASS**  
Ratio: **0.160**

**Considering z-direction:**

$V_{max} = 0.67474 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.67474 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.0060896$$

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:**

$M_{max} = 75.95 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(75.95 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.30429$$

Status: **PASS**  
Ratio: **0.300**

**Considering z-direction:**

$M_{max} = 2.6086 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.6086 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010451$$

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

REFERENCES	CALCULATIONS	RESULTS
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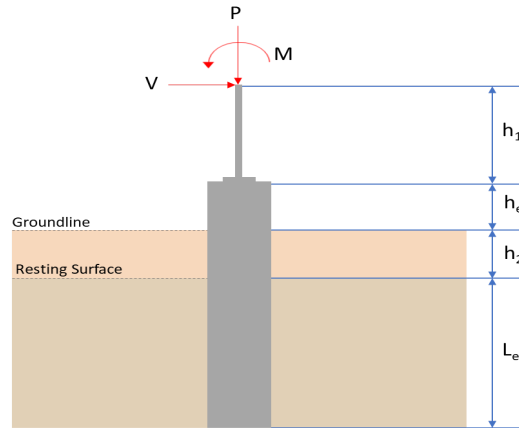
## SkyCiv Foundation Design

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

$L = 8.75$  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)	5.453	8.134
$V_x$ (kip)	-4.509	-7.510
$V_z$ (kip)	-0.424	-0.691
$M_x$ (kipft)	-1.818	-2.963
$M_z$ (kipft)	69.837	117.025

### 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,

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

$$H_o = \frac{(-4.509 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.71799 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(69.837 \text{ kipft}) + ((-4.509 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.121 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.1384 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.424 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.067516 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.818 \text{ kipft}) + ((-0.424 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28949 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.3817 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(8.1384 \text{ ft}), (2.3817 \text{ ft})]$$

$$L_{e,req} = 8.138 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(8.138 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.93006$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(5.453 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.34081 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17041$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.71799 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 11.121 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (11.121 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.121 \text{ kipft/ft})) + (4 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0328 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.121 \text{ kipft/ft})) + (3 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (11.121 \text{ kipft/ft})) + (2 \times (-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.30954 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (11.121 \text{ kipft/ft})) + ((-0.71799 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2506 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0328 \text{ ft})}{2}$$

$$p_a = 0.45246 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30954 \text{ kip/ft}^2)}{(0.45246 \text{ kip/ft}^2)}$$

$$Ratio = 0.68411$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2506 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.95287$$

Status: **PASS**  
Ratio: **0.680**

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

#### Considering z-direction:

$H_o = -0.067516 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.28949 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.28949 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.067516 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.28949 \text{ kipft/ft})) + (4 \times (-0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2536 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.28949 \text{ kipft/ft})) + (3 \times (-0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.28949 \text{ kipft/ft})) + (2 \times (-0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = -0.011809 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.28949 \text{ kipft/ft})) + ((-0.067516 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = -0.00092344 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2536 \text{ ft})}{2}$$

$$p_a = 0.46902 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.011809 \text{ kip/ft}^2)}{(0.46902 \text{ kip/ft}^2)}$$

$$Ratio = -0.025179$$

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

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

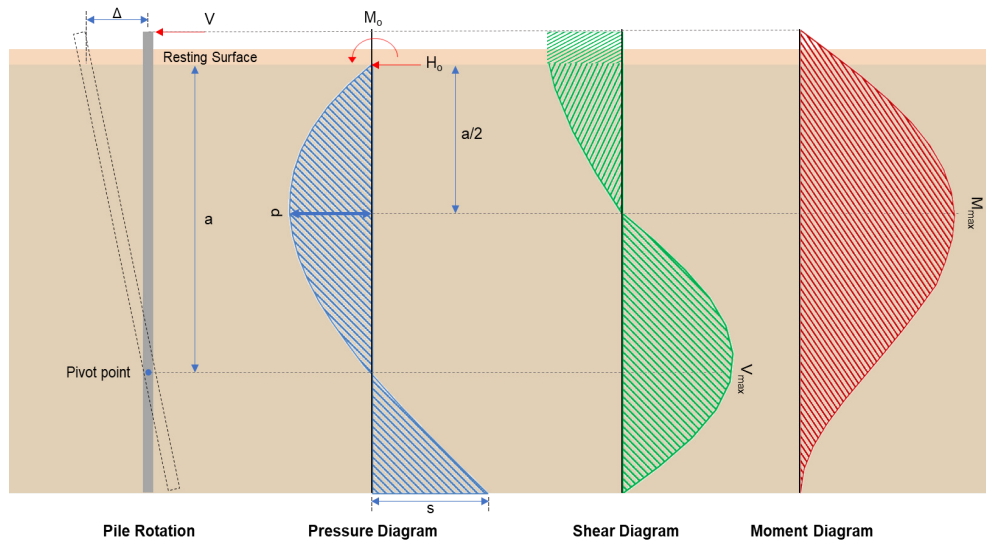
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00092344 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = -0.00070357$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-7.51 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1959 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(117.03 \text{ kipft}) + ((-7.51 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 18.635 \text{ kipft/ft}$$

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

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

$$E = \frac{(18.635 \text{ kipft/ft})}{(-1.1959 \text{ kip/ft})}$$

$$E = 15.583 \text{ ft}$$

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

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

$$a = \frac{(4 \times (18.635 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.1959 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (18.635 \text{ kipft/ft})) + (4 \times (-1.1959 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = \frac{(6 \times (18.635 \text{ kipft/ft})) + (4 \times (-1.1959 \text{ kip/ft}) \times (8.75 \text{ ft}))}{}$$

$$a = 6.0319 \text{ ft}$$

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

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

$$V_{max} = ((-1.1959 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.583 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.0319 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.583 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.0319 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 18.229 \text{ kip}$$

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

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

$$M_{max} = ((-1.1959 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[ \left( \frac{(15.583 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.583 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.583 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.0319 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 75.951 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.691 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11003 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(2.963 \text{ kipft}) + ((-0.691 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.47182 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.47182 \text{ kipft/ft})}{(-0.11003 \text{ kip/ft})}$$

$$E = 4.288 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.47182 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.11003 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.47182 \text{ kipft/ft})) + (4 \times (-0.11003 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2536 \text{ ft}$$

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

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

$$V_{max} = ((-0.11003 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.288 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.2536 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (4.288 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.2536 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.67499 \text{ kip}$$

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

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

$$M_{max} = ((-0.11003 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[ \left( \frac{(4.288 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.2536 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (4.288 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.2536 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.288 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.2536 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.6097 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.134 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.326 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.326 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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{(8.134 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0030405</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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.134 \text{ kip} \rightarrow 8134 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8134 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 18.229 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.229 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.16452$$

Status: **PASS**  
Ratio: **0.160**

**Considering z-direction:**

$V_{max} = 0.67499 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.67499 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.0060919$$

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:**

$M_{max} = 75.951 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(75.951 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.30429$$

Status: **PASS**  
Ratio: **0.300**

**Considering z-direction:**

$M_{max} = 2.6097 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.6097 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.010455$$

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

REFERENCES	CALCULATIONS	RESULTS
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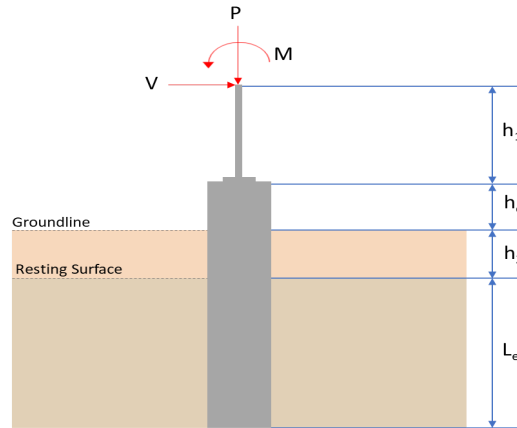
## SkyCiv Foundation Design

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

$L = 9.75$  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)	7.428	11.327
$V_x$ (kip)	-6.598	-10.986
$V_z$ (kip)	0.042	0.071
$M_x$ (kipft)	-0.169	0.289
$M_z$ (kipft)	102.833	172.713

### 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,

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

$$H_o = \frac{(-6.598 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0506 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(102.83 \text{ kipft}) + ((-6.598 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.375 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.0444 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.042 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0066879 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.169 \text{ kipft}) + ((0.042 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026911 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3946 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(9.0444 \text{ ft}), (1.3946 \text{ ft})]$$

$$L_{e,req} = 9.044 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(9.044 \text{ ft})}{(9.75 \text{ ft})}$$

$$\text{Ratio} = 0.92759$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(7.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46425 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46425 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23212$$

Status: **PASS**  
Ratio: **0.230**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(9.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -1.0506 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 16.375 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (16.375 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (16.375 \text{ kipft/ft})) + (4 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.7391 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (16.375 \text{ kipft/ft})) + (3 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]^2}{(9.75 \text{ ft})^2 \times [(3 \times (16.375 \text{ kipft/ft})) + (2 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]}$$

$$p = 0.33303 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (16.375 \text{ kipft/ft})) + ((-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]}{(9.75 \text{ ft})^2}$$

$$s = 1.4205 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.7391 \text{ ft})}{2}$$

$$p_a = 0.50543 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.33303 \text{ kip/ft}^2)}{(0.50543 \text{ kip/ft}^2)}$$

$$Ratio = 0.65889$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.75 \text{ ft})$$

$$p_s = 1.4625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.4205 \text{ kip/ft}^2)}{(1.4625 \text{ kip/ft}^2)}$$

$$Ratio = 0.97127$$

Status: **PASS**  
Ratio: **0.660**

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

#### Considering z-direction:

$H_o = 0.0066879 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.026911 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.026911 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (0.026911 \text{ kipft/ft})) + (4 \times (0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 7.0018 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.026911 \text{ kipft/ft})) + (3 \times (0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]^2}{(9.75 \text{ ft})^2 \times [(3 \times (0.026911 \text{ kipft/ft})) + (2 \times (0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]}$$

$$p = 0.0034365 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.026911 \text{ kipft/ft})) + ((0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]}{(9.75 \text{ ft})^2}$$

$$s = 0.0075127 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(7.0018 \text{ ft})}{2}$$

$$p_a = 0.52514 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0034365 \text{ kip/ft}^2)}{(0.52514 \text{ kip/ft}^2)}$$

$$Ratio = 0.0065439$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.75 \text{ ft})$$

$$p_s = 1.4625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

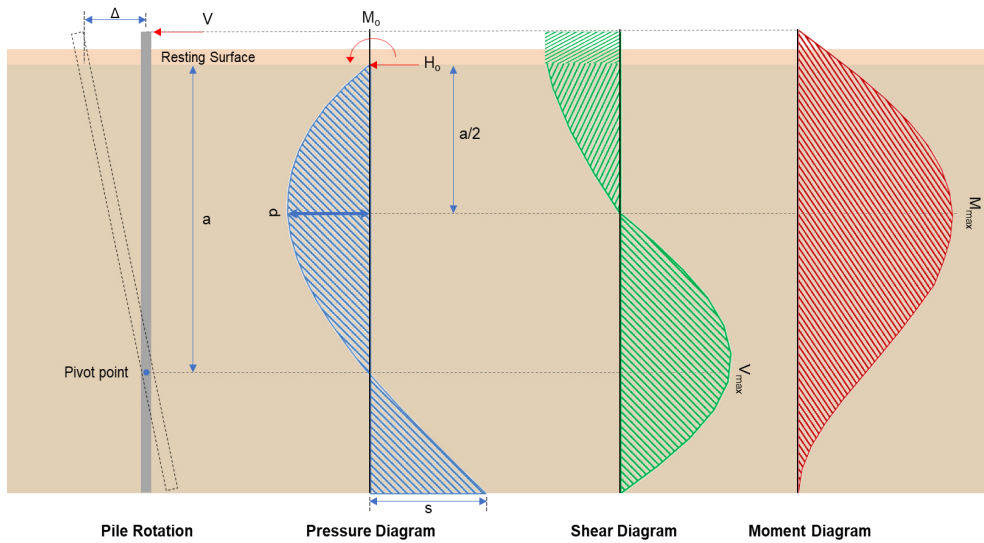
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0075127 \text{ kip/ft}^2)}{(1.4625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0051369$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-10.986 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.7494 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(172.71 \text{ kipft}) + ((-10.986 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 27.502 \text{ kipft/ft}$$

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

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

$$E = \frac{(27.502 \text{ kipft/ft})}{(-1.7494 \text{ kip/ft})}$$

$$E = 15.721 \text{ ft}$$

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

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

$$a = \frac{(4 \times (27.502 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{6 \times (27.502 \text{ kipft/ft}) + 4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft})}$$

$$a = \frac{(6 \times (27.502 \text{ kipft/ft})) + (4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft}))}{(6 \times (27.502 \text{ kipft/ft})) + (4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.7377 \text{ ft}$$

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

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

$$V_{max} = ((-1.7494 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.7377 \text{ ft})}{(9.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.7377 \text{ ft})}{(9.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 24.579 \text{ kip}$$

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

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

$$M_{max} = ((-1.7494 \text{ kip/ft}) \times (48 \text{ in}) \times (9.75 \text{ ft})) \times \left[ \left( \frac{(15.721 \text{ ft})}{(9.75 \text{ ft})} + \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 113.64 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.071 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.011306 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.289 \text{ kipft}) + ((0.071 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.046019 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.046019 \text{ kipft/ft})}{(0.011306 \text{ kip/ft})}$$

$$E = 4.0704 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.046019 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (0.011306 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (0.046019 \text{ kipft/ft})) + (4 \times (0.011306 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.9996 \text{ ft}$$

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

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

$$V_{max} = ((0.011306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.9996 \text{ ft})}{(9.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.9996 \text{ ft})}{(9.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.063622 \text{ kip}$$

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

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

$$M_{max} = ((0.011306 \text{ kip/ft}) \times (48 \text{ in}) \times (9.75 \text{ ft})) \times \left[ \left( \frac{(4.0704 \text{ ft})}{(9.75 \text{ ft})} + \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.27092 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

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

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

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

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

#### Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\left( \frac{11.327 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.22 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Min spacing of reinforcement,}</math></p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(11.327 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0042341$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 11.327 \text{ kip} \rightarrow 11327 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11327 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 24.579 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(24.579 \text{ kip})}{(111.08 \text{ kip})}$$

$$Ratio = 0.22128$$

Status: **PASS**  
Ratio: **0.220**

**Considering z-direction:**

$V_{max} = 0.063622 \text{ kip}$  - Maximum shear force in the z-direction,  
Ratio - Capacity

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

$$Ratio = \frac{(0.063622 \text{ kip})}{(111.08 \text{ kip})}$$

$$Ratio = 0.00057277$$

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:**

$M_{max} = 113.64 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(113.64 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.45527$$

Status: **PASS**  
Ratio: **0.460**

**Considering z-direction:**

$M_{max} = 0.27092 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.27092 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0010854$$

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

REFERENCES	CALCULATIONS	RESULTS
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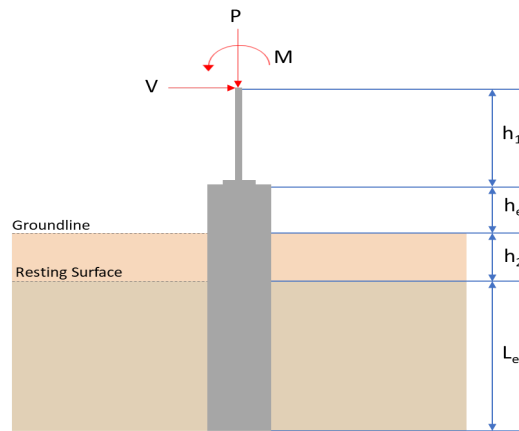
## SkyCiv Foundation Design

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

$L = 9.75$  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)	7.428	11.327
$V_x$ (kip)	-6.598	-10.986
$V_z$ (kip)	-0.042	-0.071
$M_x$ (kipft)	0.169	-0.289
$M_z$ (kipft)	102.833	172.713

### 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,

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

$$H_o = \frac{(-6.598 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0506 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(102.83 \text{ kipft}) + ((-6.598 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 16.375 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.0444 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.042 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0066879 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.169 \text{ kipft}) + ((-0.042 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026911 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1878 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(9.0444 \text{ ft}), (1.1878 \text{ ft})]$$

$$L_{e,req} = 9.044 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(9.044 \text{ ft})}{(9.75 \text{ ft})}$$

$$\text{Ratio} = 0.92759$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

$$A = b D$$

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(7.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46425 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46425 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23212$$

Status: **PASS**  
Ratio: **0.230**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(9.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.4375$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -1.0506 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 16.375 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (16.375 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (16.375 \text{ kipft/ft})) + (4 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.7391 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (16.375 \text{ kipft/ft})) + (3 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]^2}{(9.75 \text{ ft})^2 \times [(3 \times (16.375 \text{ kipft/ft})) + (2 \times (-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]}$$

$$p = 0.33303 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (16.375 \text{ kipft/ft})) + ((-1.0506 \text{ kip/ft}) \times (9.75 \text{ ft}))]}{(9.75 \text{ ft})^2}$$

$$s = 1.4205 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.7391 \text{ ft})}{2}$$

$$p_a = 0.50543 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.33303 \text{ kip/ft}^2)}{(0.50543 \text{ kip/ft}^2)}$$

$$Ratio = 0.65889$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.75 \text{ ft})$$

$$p_s = 1.4625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.4205 \text{ kip/ft}^2)}{(1.4625 \text{ kip/ft}^2)}$$

$$Ratio = 0.97127$$

Status: **PASS**  
Ratio: **0.660**

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

#### Considering z-direction:

$H_o = -0.0066879 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.026911 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.026911 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (0.026911 \text{ kipft/ft})) + (4 \times (-0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 7.0018 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.026911 \text{ kipft/ft})) + (3 \times (-0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]^2}{(9.75 \text{ ft})^2 \times [(3 \times (0.026911 \text{ kipft/ft})) + (2 \times (-0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]}$$

$$p = -0.0012291 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (0.026911 \text{ kipft/ft})) + ((-0.0066879 \text{ kip/ft}) \times (9.75 \text{ ft}))]}{(9.75 \text{ ft})^2}$$

$$s = -0.0007186 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(7.0018 \text{ ft})}{2}$$

$$p_a = 0.52514 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0012291 \text{ kip/ft}^2)}{(0.52514 \text{ kip/ft}^2)}$$

$$Ratio = -0.0023406$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9.75 \text{ ft})$$

$$p_s = 1.4625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

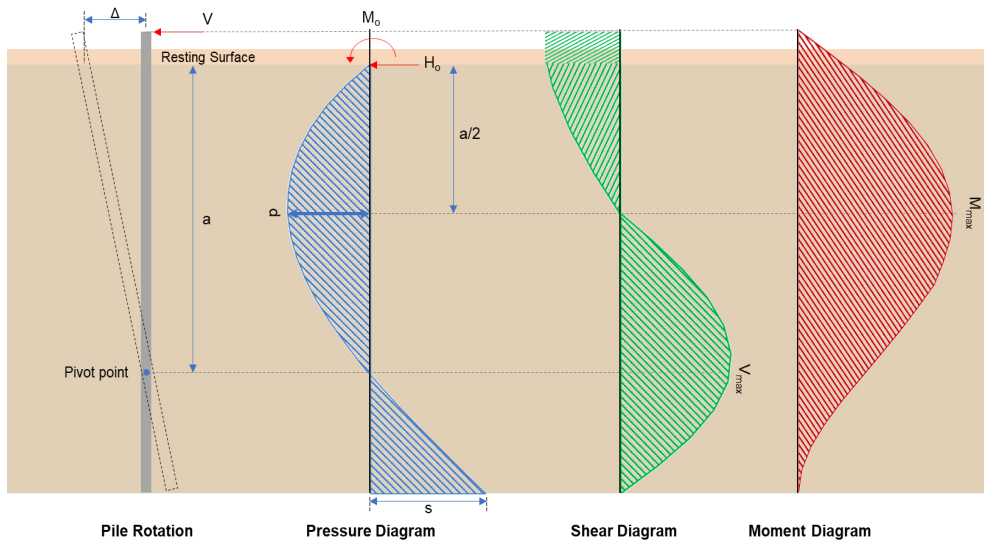
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0007186 \text{ kip/ft}^2)}{(1.4625 \text{ kip/ft}^2)}$$

$$Ratio = -0.00049135$$

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

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-10.986 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.7494 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(172.71 \text{ kipft}) + ((-10.986 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 27.502 \text{ kipft/ft}$$

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

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

$$E = \frac{(27.502 \text{ kipft/ft})}{(-1.7494 \text{ kip/ft})}$$

$$E = 15.721 \text{ ft}$$

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

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

$$a = \frac{(4 \times (27.502 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (27.502 \text{ kipft/ft})) + (4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = \frac{(-1.7494 \text{ kip/ft}) \times (48 \text{ in}) + (4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft}))}{(6 \times (27.502 \text{ kipft/ft})) + (4 \times (-1.7494 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.7377 \text{ ft}$$

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

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

$$V_{max} = ((-1.7494 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.7377 \text{ ft})}{(9.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.7377 \text{ ft})}{(9.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 24.579 \text{ kip}$$

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

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

$$M_{max} = ((-1.7494 \text{ kip/ft}) \times (48 \text{ in}) \times (9.75 \text{ ft})) \times \left[ \left( \frac{(15.721 \text{ ft})}{(9.75 \text{ ft})} + \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (15.721 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.7377 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 113.64 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.071 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.011306 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.289 \text{ kipft}) + ((-0.071 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.046019 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.046019 \text{ kipft/ft})}{(-0.011306 \text{ kip/ft})}$$

$$E = 4.0704 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.046019 \text{ kipft/ft}) \times (9.75 \text{ ft})) + (3 \times (-0.011306 \text{ kip/ft}) \times (9.75 \text{ ft})^2)}{(6 \times (0.046019 \text{ kipft/ft})) + (4 \times (-0.011306 \text{ kip/ft}) \times (9.75 \text{ ft}))}$$

$$a = 6.9996 \text{ ft}$$

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

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

$$V_{max} = ((-0.011306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.9996 \text{ ft})}{(9.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[ 4 \times \left( \frac{3 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.9996 \text{ ft})}{(9.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.063622 \text{ kip}$$

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

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

$$M_{max} = ((-0.011306 \text{ kip/ft}) \times (48 \text{ in}) \times (9.75 \text{ ft})) \times \left[ \left( \frac{(4.0704 \text{ ft})}{(9.75 \text{ ft})} + \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right) \right. \\ \left. - \left[ \left( \frac{4 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 3 \right) \times \left( \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (4.0704 \text{ ft})}{(9.75 \text{ ft})} + 2 \right) \times \left( \frac{(6.9996 \text{ ft})}{2 \times (9.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.27092 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(11.327 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.22 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;"><math>Ratio = 0.96556</math></p> <p><math>s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]</math></p> <p><math>s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]</math></p> <p><math>s_{rebar} = 1.5 \text{ in}</math></p> <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]</math></p> <p><math>s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]</math></p> <p><math>s_{ties} = 10 \text{ in}</math></p> <p><b>Summary:</b></p> <p style="text-align: center;">Main reinforcement: <b>14 - #5 (0.625 in)</b> Ties: <b>#3(0.375 in) - 10 in</b></p>	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>
<p>22.4.2.2</p>	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> <p style="text-align: center;"><math>\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k 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{(11.327 \text{ kip})}{(2675.2 \text{ kip})}</math></p> <p style="text-align: center;"><math>Ratio = 0.0042341</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 = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]</math></p> <p style="text-align: center;"><math>\lambda_s = 0.64282</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>,</p> <p><math>V_{c,max}</math> - Max shear strength of concrete</p> <p style="text-align: center;"><math>V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d</math></p> <p style="text-align: center;"><math>V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})</math></p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 11.327 \text{ kip} \rightarrow 11327 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11327 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.08 \text{ kip}$$

**Considering x-direction:**

$V_{max}$  = 24.579 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(24.579 \text{ kip})}{(111.08 \text{ kip})}$$

$$Ratio = 0.22128$$

Status: **PASS**  
Ratio: **0.220**

**Considering z-direction:**

$V_{max} = 0.063622 \text{ kip}$  - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.063622 \text{ kip})}{(111.08 \text{ kip})}$$

$$Ratio = 0.00057277$$

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:**

$M_{max} = 113.64 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(113.64 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.45527$$

Status: **PASS**  
Ratio: **0.460**

**Considering z-direction:**

$M_{max} = 0.27092 \text{ kipft}$  - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.27092 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0010854$$

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