

# Your Project Calculations



Project Name: MTSOLAR\_EDD7CHE6LH6H

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=MTSOLAR\\_EDD7CHE6LH6H&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/4\\_2023](https://platform.skyciv.com/structural?preload_name=MTSOLAR_EDD7CHE6LH6H&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=10cvKPic4gM481V1xTF4PO2ngZaiAkdMTZ3OqBMeOqj26KLk6U67KZYj6BOUaifZ](https://platform.skyciv.com/structural-viewer?project_id=10cvKPic4gM481V1xTF4PO2ngZaiAkdMTZ3OqBMeOqj26KLk6U67KZYj6BOUaifZ)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	3P-19.75-6TOP-HD-24-L-4Hx7W-04CE
<b>Duty Classification:</b>	HD
<b>Module Width:</b>	41.10 in
<b>Module Length:</b>	87.20in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	7
<b>Total Number of Modules:</b>	28
<b>Desired Tilt Angle:</b>	31
<b>Front Edge Clearance:</b>	7
<b>Total Array Height at Tilt:</b>	14.10 ft
<b>Total Frame Length:</b>	51.00 ft
<b>Frame Weight:</b>	2570 lbs
<b>Array Dimensions N/S:</b>	13.87 ft
<b>Array Dimensions E/W:</b>	51.45 ft
<b>Rail Length:</b>	166.40 in
<b>Rail Spacing:</b>	3.68 ft
<b>Rail Check:</b>	PASS (39% utilized)

## Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 80
<b>Pole Length above Grade:</b>	10.57 ft
<b>Number of Poles:</b>	3
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.00 ft Pile 2: 6.50 ft Pile 3: 6.00 ft
<b>Foundation Volume:</b>	10.963 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.894647 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	495 Tri Dale Ln, Newcastle, CA 95658, USA
<b>Wind Speed:</b>	95 mph
<b>Snow Load:</b>	10 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.004289 ksf



### Design Disclaimer

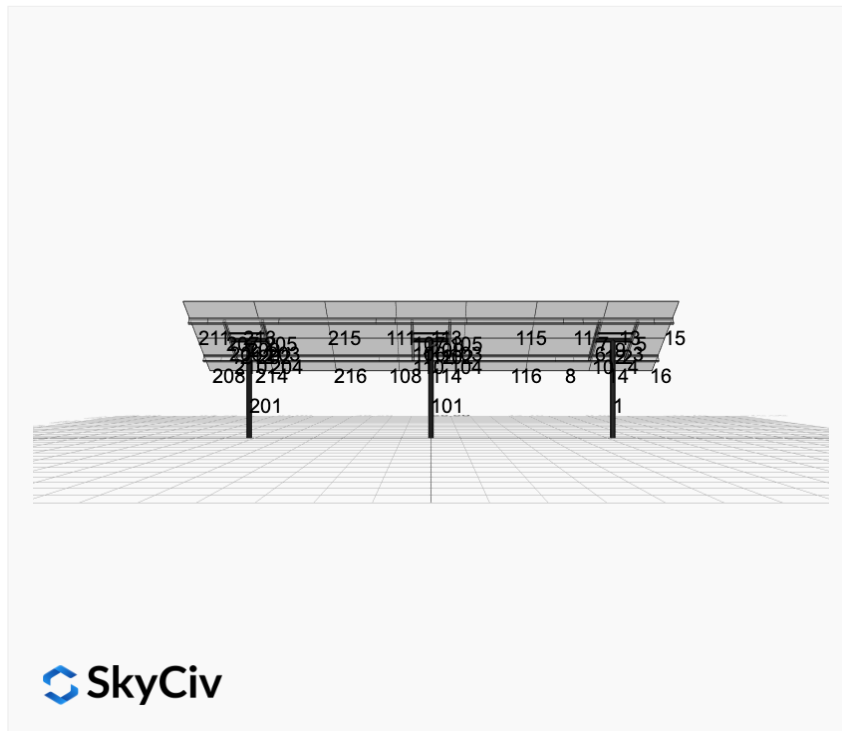
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

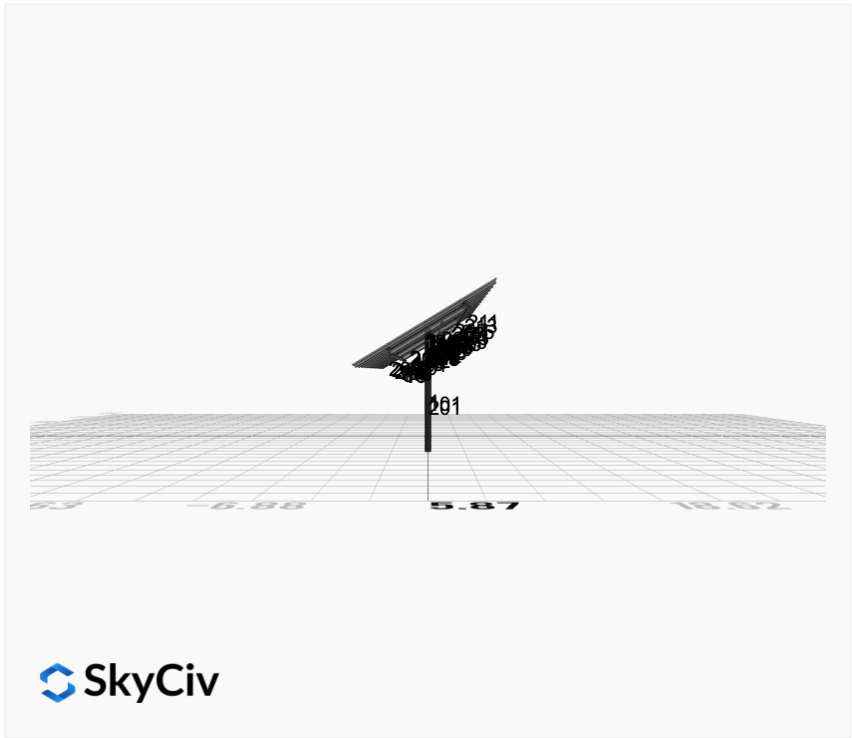
### AutoDesigner Input

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  "module_length": 87.2,
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  "number_columns": 7,
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  "adjuster_section": "2_40",
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  "pole_override": "auto",
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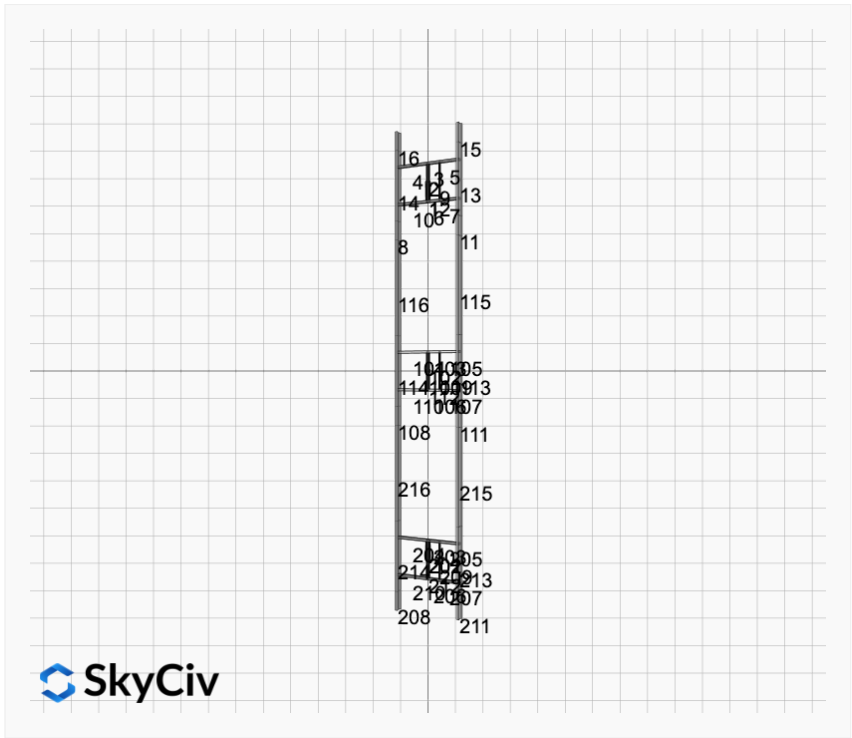
### Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent

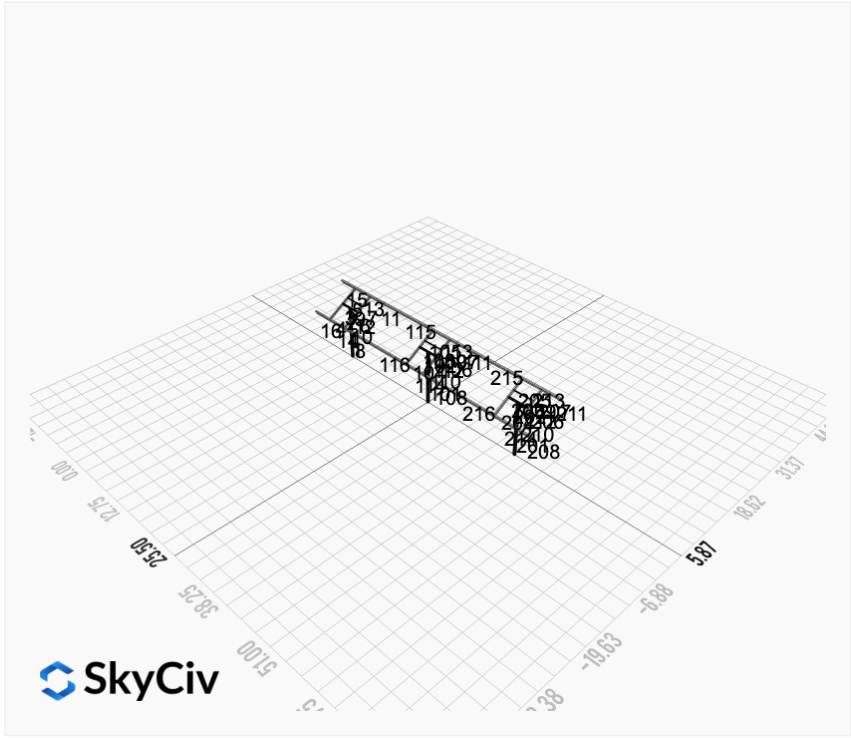




 SkyCiv



 SkyCiv



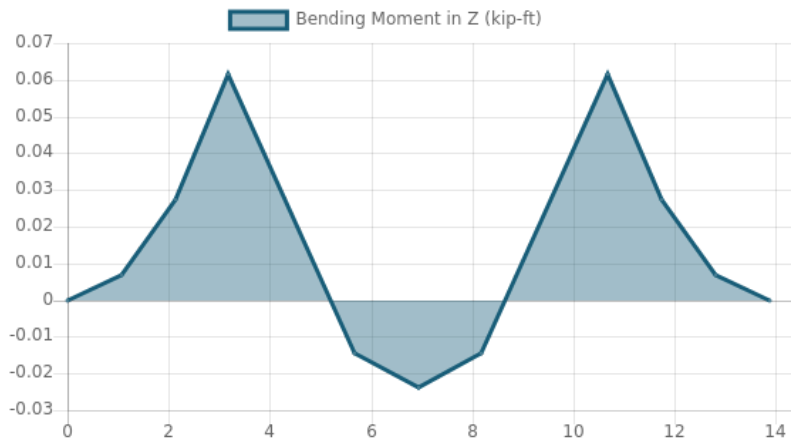
**Rail Design Check**

**Rail Length:** 13.866666666666667 ft  
**Additional Restraints Required:** None  
**Tributary Width:** 3.6750000000000003 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Snow (X):** 0.0135 kip/ft  
**Snow (Y):** -0.0081 kip/ft  
**Wind uplift Case A:** 0.0916 kip/ft  
**Wind uplift Case A:** 0.0916 kip/ft  
**Wind uplift Case B (X):** 0.0000 kip/ft  
**Wind uplift Case B (Y):** 0.1265 kip/ft

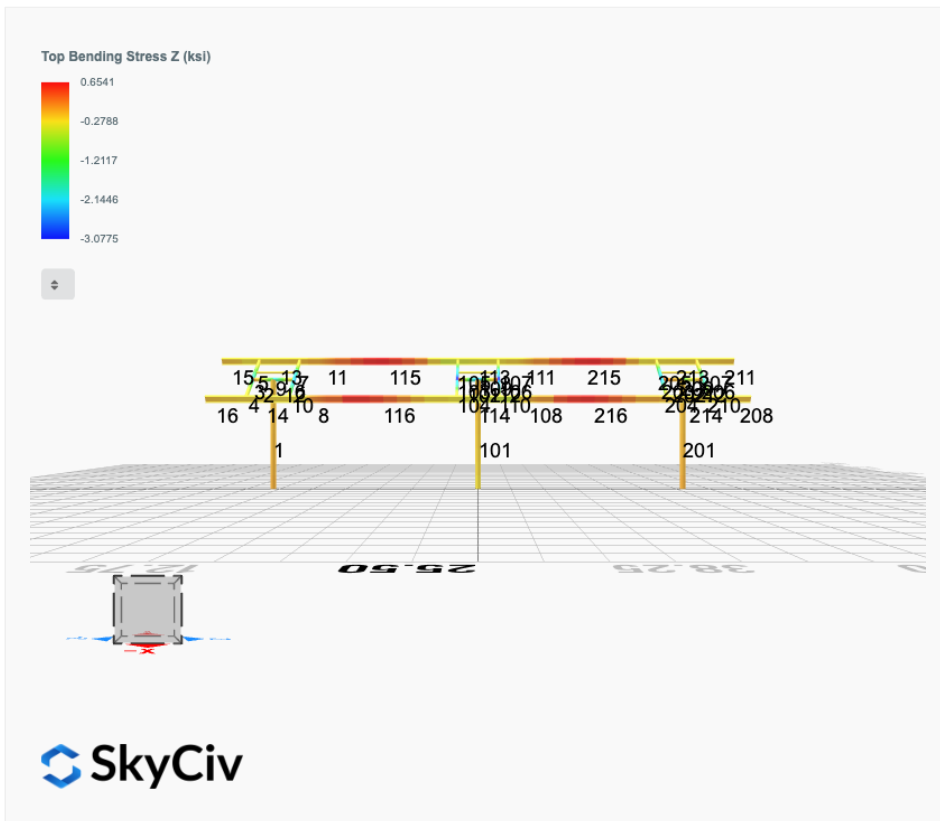
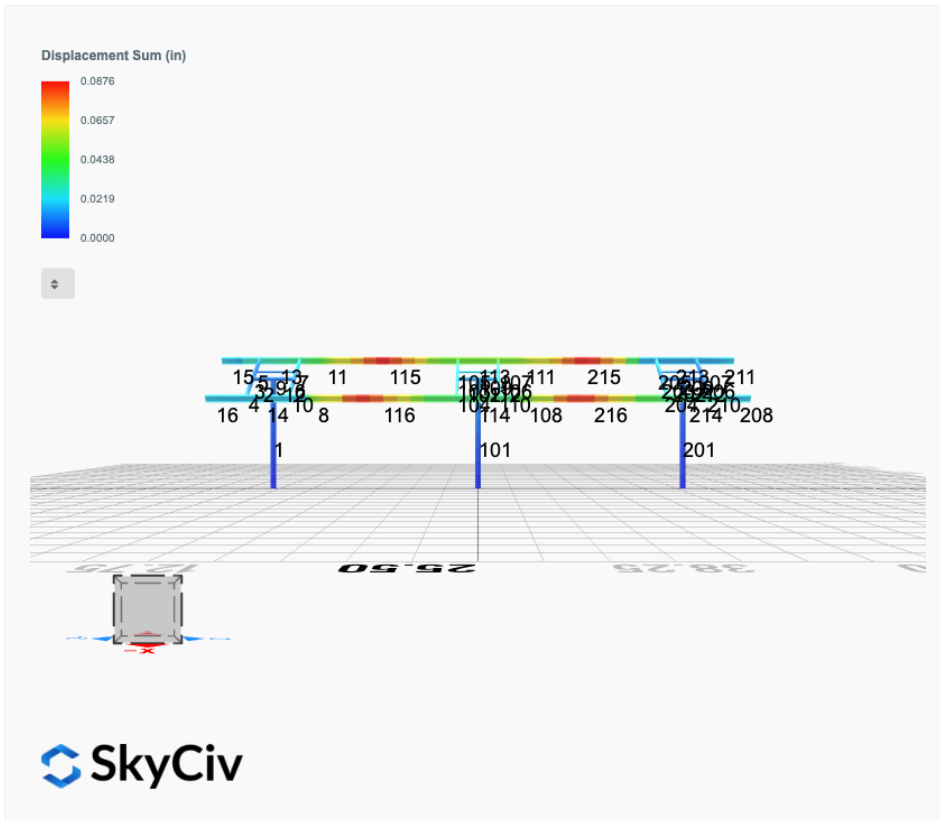


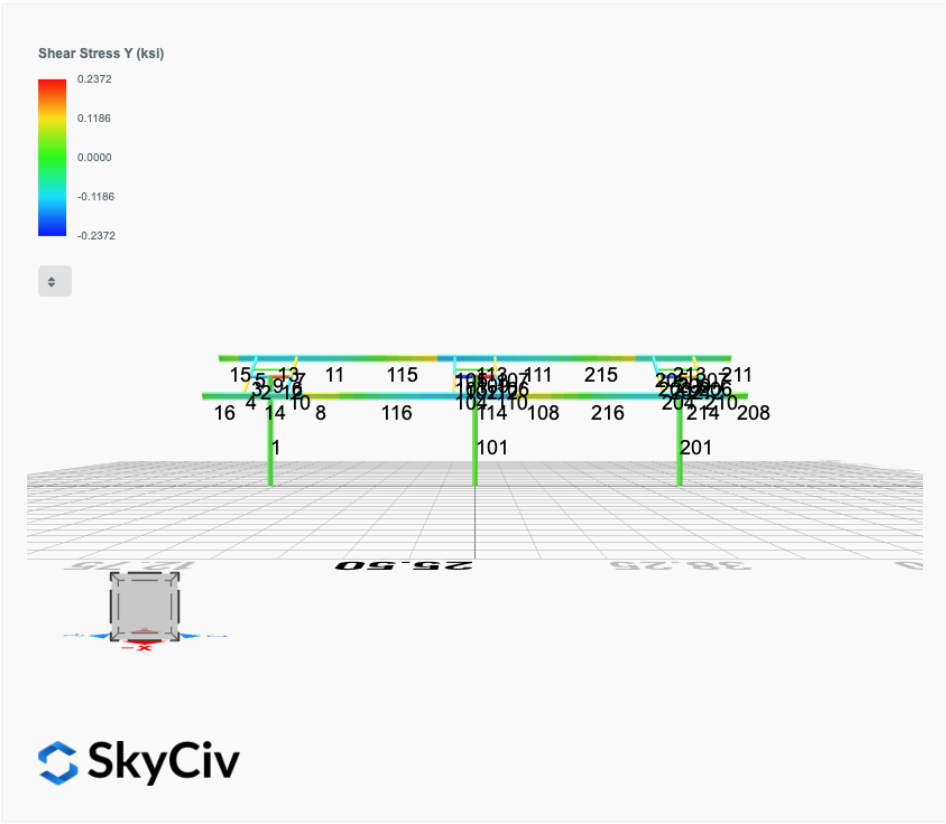
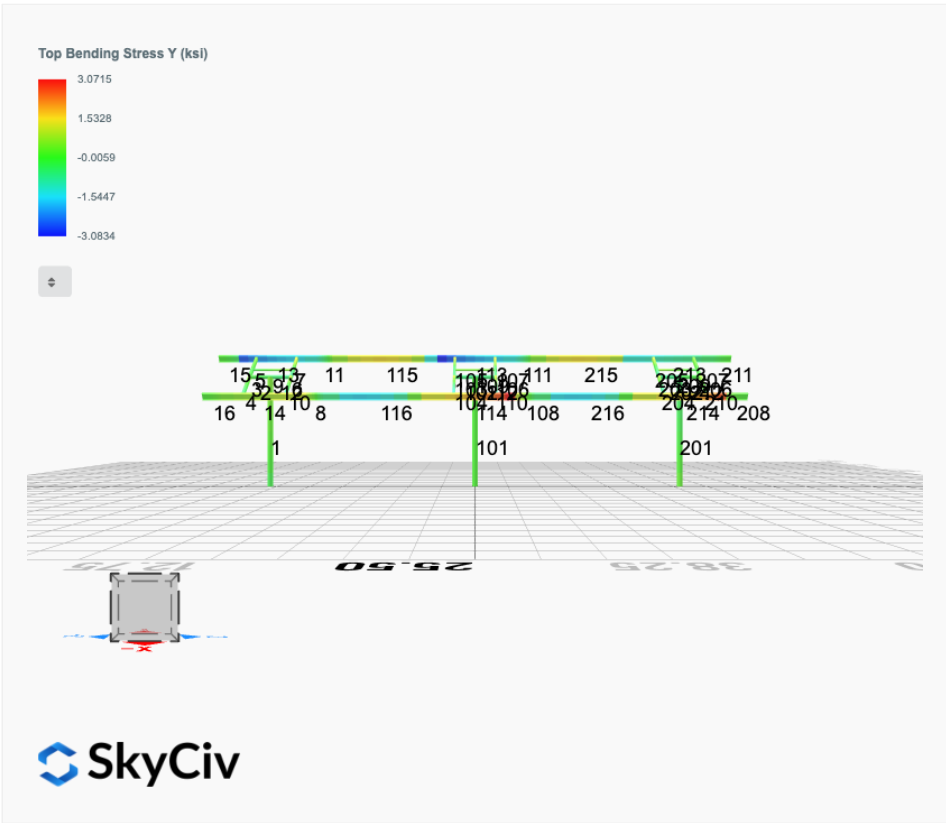
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	13.43375579	0.389	PASS
Material Yield	34.5	13.43375579	0.389	PASS
Material Strength	37	13.43375579	0.363	PASS

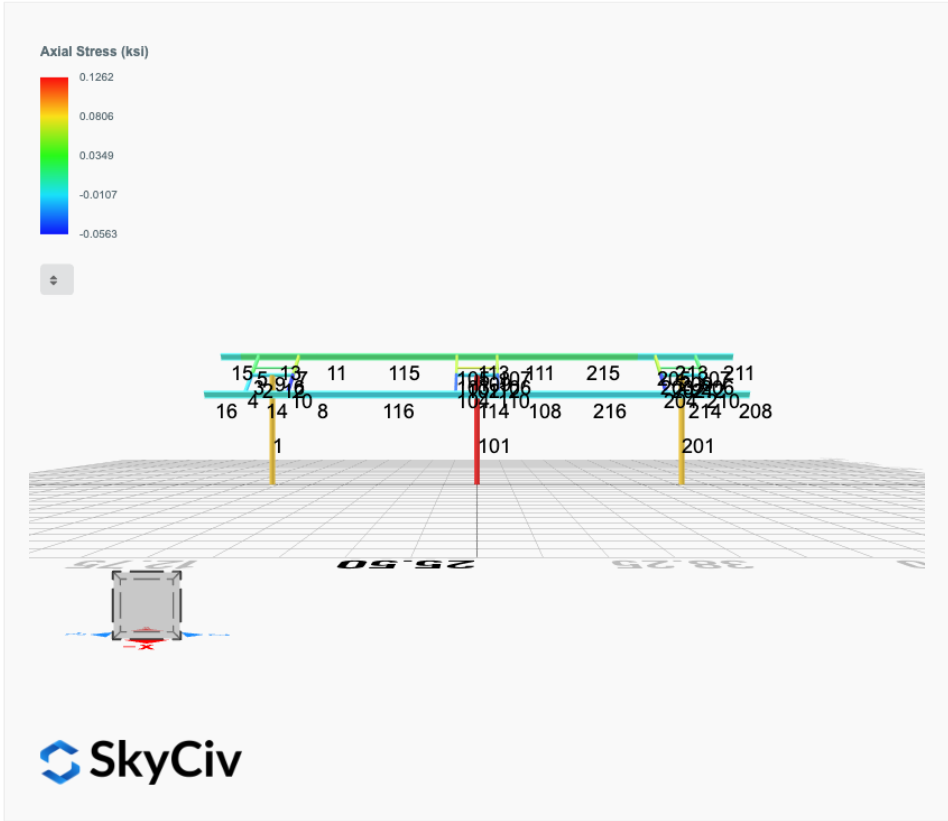
**Member 1, ULS: 1. 1.4D**



# FEM Results (Envelope Worst Case for each member)







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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0214	1.7871	0.0699	0.2264	-0.0518	-0.1679
ULS: 2. D + L	0.0214	1.7871	0.0699	0.2264	-0.0518	-0.1679
ULS: 3. D + (S or Lr or R)	0.0344	2.5565	0.1123	0.3635	-0.0832	-0.2845
ULS: 3. D + (S or Lr or R)	0.0214	1.7871	0.0699	0.2264	-0.0518	-0.1679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0312	2.3641	0.1017	0.3292	-0.0753	-0.2554
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0214	1.7871	0.0699	0.2264	-0.0518	-0.1679
ULS: 5b. D + 0.7E	0.0214	1.7871	0.0699	0.2264	-0.0518	-0.1679
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0312	2.3641	0.1017	0.3292	-0.0753	-0.2554
ULS: 8. 0.6D + 0.7E	0.0129	1.0723	0.0420	0.1358	-0.0311	-0.1007
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9279	4.9473	0.3208	1.0166	-0.5380	21.3293
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.9279	4.9473	0.3208	1.0166	-0.5380	21.3293
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6811	-0.9082	-0.1376	-0.4251	0.3502	-17.6012
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4337	-0.4793	-0.1288	-0.3970	0.3400	-21.5942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4308	4.7343	0.2898	0.9219	-0.4400	15.8675
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4308	4.7343	0.2898	0.9219	-0.4400	15.8675
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2759	0.3426	-0.0540	-0.1594	0.2261	-13.3303
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0904	0.6643	-0.0474	-0.1383	0.2185	-16.3251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4405	4.1573	0.2581	0.8191	-0.4164	15.9550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4405	4.1573	0.2581	0.8191	-0.4164	15.9550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2662	-0.2344	-0.0857	-0.2623	0.2497	-13.2429
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0806	0.0873	-0.0791	-0.2412	0.2421	-16.2376
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9364	4.2325	0.2928	0.9261	-0.5173	21.3965
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9364	4.2325	0.2928	0.9261	-0.5173	21.3965
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6725	-1.6231	-0.1656	-0.5157	0.3709	-17.5340
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4251	-1.1942	-0.1568	-0.4876	0.3607	-21.5270

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	7.7984
Shear X	-3.2488
Shear Z	0.5265
Moment X	1.6696
Moment Y (Twist)	0.8945
Moment Z	36.4930

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.9473
Shear X	-1.9364
Shear Z	0.3208
Moment X	1.0166
Moment Y (Twist)	0.5380
Moment Z	21.5942

## 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	-0.0429	2.2692	0.0000	0.0000	0.0000	0.4148
ULS: 2. D + L	-0.0429	2.2692	0.0000	0.0000	0.0000	0.4148
ULS: 3. D + (S or Lr or R)	-0.0688	3.3302	0.0000	-0.0000	0.0000	0.6510
ULS: 3. D + (S or Lr or R)	-0.0429	2.2692	0.0000	0.0000	0.0000	0.4148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0624	3.0649	0.0000	-0.0000	0.0000	0.5920
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0429	2.2692	0.0000	0.0000	0.0000	0.4148
ULS: 5b. D + 0.7E	-0.0429	2.2692	0.0000	0.0000	0.0000	0.4148

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0624	3.0649	0.0000	-0.0000	0.0000	0.5920
ULS: 8. 0.6D + 0.7E	-0.0257	1.3615	0.0000	0.0000	0.0000	0.2489
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5726	6.6473	0.0000	-0.0000	0.0000	27.4680
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5726	6.6473	0.0000	-0.0000	0.0000	27.4680
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1303	-1.4811	0.0000	-0.0000	0.0000	-21.4484
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7097	-0.8154	0.0000	-0.0000	0.0000	-25.4840
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9597	6.3485	0.0000	-0.0000	0.0000	20.8819
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9597	6.3485	0.0000	-0.0000	0.0000	20.8819
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5675	0.2522	0.0000	-0.0000	0.0000	-15.8054
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2521	0.7515	0.0000	-0.0000	0.0000	-18.8321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9402	5.5528	0.0000	-0.0000	0.0000	20.7047
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9402	5.5528	0.0000	-0.0000	0.0000	20.7047
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5870	-0.5436	0.0000	-0.0000	0.0000	-15.9826
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2716	-0.0443	0.0000	-0.0000	0.0000	-19.0093
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5554	5.7396	0.0000	-0.0000	0.0000	27.3021
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5554	5.7396	0.0000	-0.0000	0.0000	27.3021
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1474	-2.3888	0.0000	-0.0000	0.0000	-21.6143
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7269	-1.7231	0.0000	-0.0000	0.0000	-25.6499

#### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.5461
Shear X	-4.2746
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	46.3772

#### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6473
Shear X	-2.5726
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	27.4680

#### 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	0.0214	1.7871	-0.0699	-0.2264	0.0518	-0.1679
ULS: 2. D + L	0.0214	1.7871	-0.0699	-0.2264	0.0518	-0.1679
ULS: 3. D + (S or Lr or R)	0.0344	2.5565	-0.1123	-0.3635	0.0832	-0.2845
ULS: 3. D + (S or Lr or R)	0.0214	1.7871	-0.0699	-0.2264	0.0518	-0.1679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0312	2.3641	-0.1017	-0.3292	0.0753	-0.2554
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0214	1.7871	-0.0699	-0.2264	0.0518	-0.1679
ULS: 5b. D + 0.7E	0.0214	1.7871	-0.0699	-0.2264	0.0518	-0.1679
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0312	2.3641	-0.1017	-0.3292	0.0753	-0.2554
ULS: 8. 0.6D + 0.7E	0.0129	1.0723	-0.0420	-0.1358	0.0311	-0.1007
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9279	4.9473	-0.3208	-1.0167	0.5380	21.3293
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.9279	4.9473	-0.3208	-1.0167	0.5380	21.3293
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6811	-0.9082	0.1376	0.4251	-0.3502	-17.6012
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4337	-0.4793	0.1288	0.3970	-0.3400	-21.5942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4308	4.7343	-0.2898	-0.9219	0.4400	15.8675
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4308	4.7343	-0.2898	-0.9219	0.4400	15.8675
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2759	0.3426	0.0540	0.1594	-0.2261	-13.3303
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0904	0.6643	0.0474	0.1383	-0.2185	-16.3251

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4405	4.1573	-0.2581	-0.8191	0.4164	15.9550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.4405	4.1573	-0.2581	-0.8191	0.4164	15.9550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2662	-0.2344	0.0857	0.2622	-0.2497	-13.2428
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0806	0.0873	0.0791	0.2412	-0.2421	-16.2376
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9364	4.2325	-0.2928	-0.9261	0.5173	21.3965
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.9364	4.2325	-0.2928	-0.9261	0.5173	21.3965
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6725	-1.6231	0.1656	0.5157	-0.3709	-17.5340
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.4251	-1.1942	0.1568	0.4876	-0.3607	-21.5270

### Worst Case Reactions LRFD

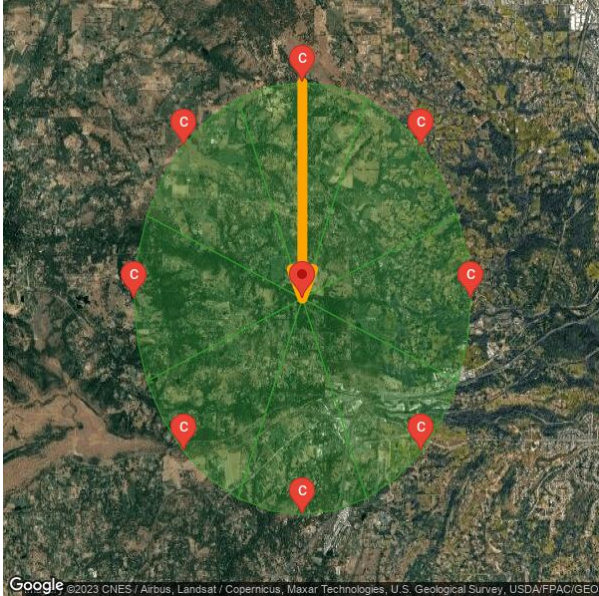
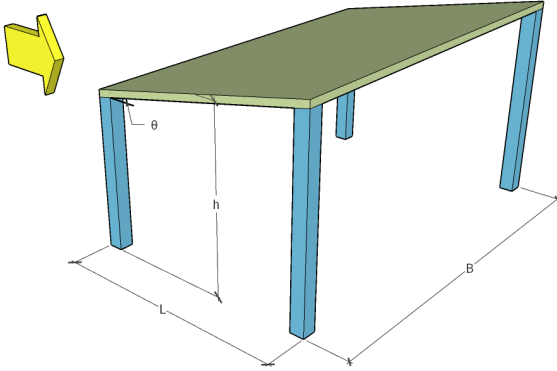
These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.  
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.7984
Shear X	-3.2488
Shear Z	-0.5265
Moment X	-1.6698
Moment Y (Twist)	0.8946
Moment Z	36.4938

### Worst Case Reactions ASD

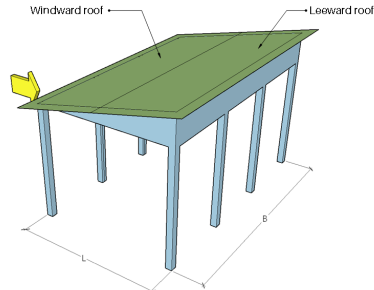
These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.  
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	4.9473
Shear X	-1.9364
Shear Z	-0.3208
Moment X	-1.0167
Moment Y (Twist)	0.5380
Moment Z	21.5942

REFERENCES	CALCULATIONS	RESULTS																
	<p style="text-align: center;"><b>Wind Load Calculations based on ASCE 7-16</b></p> <p><b>Design Information :</b>  Project Name : MTSOLAR_EDD7CHE6LH6H  Client :  Designer : MT SKYCIV AutoDesigner  Company : MT Solar  Units : Imperial  Notes : Snow loads based on monoslope structure</p> <p><b>Project Data</b>  The structure is located in <b>495 Tri Dale Ln, Newcastle, CA 95658</b> categorized as <b>Exposure C</b> (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Moreover, the structure is classified as <b>Risk Category I</b>. The location is elevated at <b>650.15 ft</b> above mean sea level.</p>  <p style="text-align: center;"><b>Figure 1. Site location.</b></p> <table border="1" data-bbox="609 1142 986 1328"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Building Length, <math>L</math></td> <td>11.74 ft</td> </tr> <tr> <td>Building Width, <math>B</math></td> <td>51.00 ft</td> </tr> <tr> <td>Mean Roof Height, <math>h</math></td> <td>10.57 ft</td> </tr> <tr> <td>Roof Profile</td> <td>Open Monoslope</td> </tr> <tr> <td>Roof Pitch Angle, <math>\theta</math></td> <td>31.00°</td> </tr> <tr> <td>Structure Type</td> <td>Main Wind Force Resisting System (MWFRS)</td> </tr> <tr> <td>Wind Blockage</td> <td>Empty Under</td> </tr> </tbody> </table>  <p style="text-align: center;"><b>Figure 2. Building parameters.</b></p>	Parameter	Value	Building Length, $L$	11.74 ft	Building Width, $B$	51.00 ft	Mean Roof Height, $h$	10.57 ft	Roof Profile	Open Monoslope	Roof Pitch Angle, $\theta$	31.00°	Structure Type	Main Wind Force Resisting System (MWFRS)	Wind Blockage	Empty Under	
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Wind Blockage	Empty Under																	
<p>Figure 26.5-1</p>	<p><b>Basic Wind Speed, <math>V</math></b>  Wind speed for the address is <b>95 mph (defined by the user)</b> for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.</p>	<p><math>V = 95</math> mph (<b>defined by the user</b>)</p>																
<p>Table 207A.6-1</p>	<p><b>Wind Directionality Factor, <math>K_d</math></b>  <math>K_d = 0.85</math> - Wind Directionality Factor  For buildings</p>	<p><math>K_d = 0.85</math></p>																
<p>Section 26.8.1</p>	<p><b>Topographic Factor, <math>K_{zt}</math></b>  <math>K_{zt} = 1</math> - Topographic Factor  For the selected wind source direction, either the terrain is relatively a flat surface or the structure is outside the local topographic zones.</p>	<p><math>K_{zt} = 1</math></p>																

	<b>For calculating the hill-shape multiplier, the detected topography for the selected wind source direction is Flat.</b>																																																																										
Section 26.9	<p><b>Ground Elevation Factor, <math>K_e</math></b></p> <p><math>K_e</math> - Ground Elevation Factor</p> $K_e = e^{-0.000862 E}$ $K_e = 0.97674$ <p>Where <math>E</math> = Site Elevation = 650.15 ft</p>																																																																										
Section 26.9	$K_e = 0.97674$ - Ground Elevation Factor		$K_e = 0.97674$																																																																								
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Section 26.10.2	<p><b>Velocity Pressure, <math>q_h</math></b></p> <p>For the selected wind source direction.</p> <p><math>q_h</math> - Velocity Pressure at h</p> $q_h = 0.00256 K_{z,h} K_{zt} K_d K_e V^2$ $q_h = 16.283 \text{ psf}$ <p>Where <math>K_{z,h} = 0.84888</math>  <math>K_{zt}</math> = Topographic Factor = 1  <math>K_d</math> = Wind Directionality Factor = 0.85  <math>V</math> = Basic Wind Speed = 95 mi/h  <math>K_e</math> = Ground Elevation Factor = 0.97674</p>																																																																										
Section 26.8	<p><b>Velocity Pressure for All Directions</b></p> <p><math>K_{zt}</math> - Topographic Factor</p> $K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$																																																																										
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Figure 27.3-4 to 27.3-7	<p><b>Net Pressure Coefficients, <math>C_N</math></b></p> <p>The net pressure coefficients, <math>C_N</math>, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.</p>																																																																										
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Section 26.11	<p><b>Gust Effect Factor, <math>G</math></b></p> <p><math>G = 0.85</math> - Gust Effect Factor</p> <p>The structure is assumed to be rigid.</p>		$G = 0.85$																																																																								
Section 27.3.2	<p><b>Design Wind Pressures (MWFRS)</b></p> <p><math>p</math> - Design Wind Pressure</p> <p>For open buildings</p> $p = q_h \times G \times C_N$ <p>For Wind Pressure - 0°</p>																																																																										

Direction	Surface	$p$ Case A (psf)	$p$ Case B (psf)
0	Windward	-24.913	-34.417
	Leeward	-24.913	-7.105
180	Windward	29.065	36.170
	Leeward	29.250	14.025



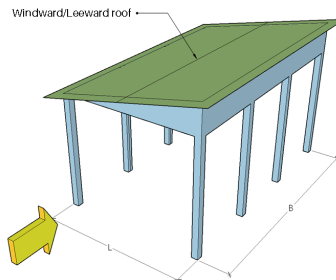
Wind along L - 0°

Service Wind Pressure - 0°/180°

Direction	Surface	$p$ Case A (psf)	$p$ Case B (psf)
0	Windward	-14.948	-20.650
	Leeward	-14.948	-4.263
180	Windward	17.439	21.702
	Leeward	17.550	8.415

For Wind Pressure - 90°

Direction	Surface	$p$ Case A (psf)	$p$ Case B (psf)
90	≤ h from windward edge	-11.072	11.072
	h to 2h from windward edge	-8.304	6.920
	> 2h from windward edge	-4.152	4.152



Wind along B - 90°

Service Wind Pressure - 90°

Direction	Surface	$p$ Case A (psf)	$p$ Case B (psf)
90	≤ h from windward edge	-6.643	6.643
	h to 2h from windward edge	-4.983	4.152
	> 2h from windward edge	-2.491	2.491

Section 27.3.2  
Section 28.3.5

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume  $K_S = 1.0$  and should be adjusted and be reduced based on the actual solidity ratio  $\phi$  and number of frames  $n$  - See Figure 28.3-2.

Section 27.3.2  
Section 28.3.5

$p$  - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$

Section 27.3.2  
Section 28.3.5

$K_B$  - Frame Width Factor

For  $L < 100ft$ ,  $K_B = 1.8 - 0.01L$ . Otherwise,  $K_B = 0.8$ .

$$K_B = 1.8 - 0.01 * L \leq 0.8$$

$$K_B = 1.6826$$

Where  $L$  = Building Length = 11.743 ft

Section 28.3.5  $K_S$  - Shielding Factor

$$K_S = 0.6 + 0.073 \times (n - 1) + (1.25 \times \phi^{1.8})$$

Section 28.3.5  $K_S = 1$  - Shielding Factor

Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio  $\phi$  and number of frames  $n$ .

Figure 28.3-1  
Section 28.3.5  $(GC_{pf})_{windward} = 0.4$

Using Zone 5 from Figure 28.3-1

Figure 28.3-1  
Section 28.3.5  $(GC_{pf})_{leeward} = -0.29$

Using Zone 6 from Figure 28.3-1

Section 27.3.2  
Section 28.3.5  $p$  - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)


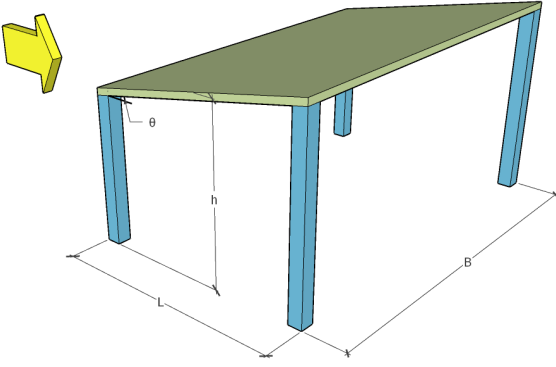
$$p = q_h [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] K_B K_S$$

$$p = 18.904 \text{ psf}$$

$$K_S = 1$$

$$(GC_{pf})_{windward} = 0.4$$

$$(GC_{pf})_{leeward} = -0.29$$

REFERENCES	CALCULATIONS	RESULTS												
	<p style="text-align: center;"><b>Snow Load Detailed Calculations based on ASCE 7-16</b></p> <p><b>Design Information :</b></p> <p>Project Name : MTSOLAR_EDD7CHE6LH6H  Client :  Designer : MT_SKYCIV AutoDesigner  Company : MT Solar  Units : Imperial  Notes : Snow loads based on monoslope structure</p> <p><b>Project Data</b></p> <p>The structure is located in <b>495 Tri Dale Ln, Newcastle, CA 95658</b> categorized as <b>Risk Category I</b>. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at <b>650 ft</b> above mean sea level.</p>  <p style="text-align: center;"><b>Figure 1. Site location.</b></p> <p>Additional details of the structure are shown in Table below and illustrated in Figure 2:</p> <table border="1" data-bbox="592 1182 1003 1417"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Building Length, <math>L</math></td> <td>11.743 ft</td> </tr> <tr> <td>Building Width, <math>B</math></td> <td>51.000 ft</td> </tr> <tr> <td>Mean Roof Height, <math>h</math></td> <td>10.571 ft</td> </tr> <tr> <td>Roof Profile</td> <td>Open Monoslope</td> </tr> <tr> <td>Roof Pitch Angle, <math>\theta</math></td> <td>31.000°</td> </tr> </tbody> </table>  <p style="text-align: center;"><b>Figure 2. Building parameters.</b></p>	Parameter	Value	Building Length, $L$	11.743 ft	Building Width, $B$	51.000 ft	Mean Roof Height, $h$	10.571 ft	Roof Profile	Open Monoslope	Roof Pitch Angle, $\theta$	31.000°	
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Roof Profile	Open Monoslope													
Roof Pitch Angle, $\theta$	31.000°													
<p>Section 7.2 of ASCE 7</p>	<p><b>Ground Snow Load, <math>p_g</math></b></p> <p>The ground snow load, <math>p_g</math>, for the site location is <b>10 psf (defined by the user)</b> at elevation 650.15 ft above mean sea level based on Section 7.2 of ASCE 7</p>	<p><math>p_g = 10</math> psf (defined by the user)</p>												

	based on Section 7.2 of ASCE 7.	by the user
Table 7-2 Section 7.3.1 of ASCE 7	<p><b>Exposure Factor, <math>C_e</math></b></p> <p>The exposure factor, <math>C_e</math>, for the structure is equal 0.90 as the terrain is categorized as <b>Exposure C</b> with exposure condition specified as <b>Fully Exposed</b> based on Table 7-2 Section 7.3.1 of ASCE 7.</p>	$C_e = 0.90$
Table 7-3 Section 7.3.2 of ASCE 7	<p><b>Thermal Factor, <math>C_t</math></b></p> <p>Since the thermal condition of the structure is categorized as "<b>Unheated and open air structures</b>," the corresponding thermal factor, <math>C_t</math>, is equal 1.20 based on Table 7-3 Section 7.3.2 of ASCE 7.</p>	$C_t = 1.20$
Table 1.5-2 of Chapter 1 ASCE 7	<p><b>Importance Factor, <math>I_s</math></b></p> <p>Since the structure is classified Risk Category I, the Importance Factor, <math>I_s</math>, is equal to <b>0.8</b>.</p>	$I_s = 0.80$
Equation 7.3-1 of Section 7.3 ASCE 7	<p><b>Flat Roof Snow Load, <math>p_f</math></b></p> <p>The flat roof snow load, <math>p_f</math>, (psf) is calculated using the Equation 7.3-1:</p> $p_f = 0.7C_eC_tI_s p_g$ $p_f = 0.7(0.90)(1.20)(0.80)(10.00) = 6.05psf$	$p_f = 6.05 \text{ psf}$
Section 7.10 ASCE 7	<p><b>Rain-on-snow Surcharge Load, <math>p_r</math></b></p> <p>The rain-on-snow surcharge load, <math>p_r</math>, is equal to 0.00 psf since <math>p_g \leq 20</math> psf but not zero. This is in addition to the sloped roof (balanced) load case.</p>	$p_r = 0.00 \text{ psf}$
Equation 7.7-1 of ASCE 7	<p><b>Snow Density, <math>\gamma</math></b></p> <p>The snow density, <math>\gamma</math>, is calculated using Equation 7.7-1 of ASCE 7 as:</p> $\gamma = 0.13p_g + 14 \leq 30 = 0.13(10.00) + 14 \leq 30$ $\gamma = 15.30pcf$	$\gamma = 15.30pcf$
Section 7.4 ASCE 7	<p><b>Roof Slope Factor (Balanced), <math>C_s</math></b></p> <p>Since the roof is classified as cold roof (<math>C_t &gt; 1.0</math>), the corresponding roof slope factor, <math>C_s</math>, is equal to 0.709 based on Figure 7.2c where <math>\theta = 31.00^\circ</math>.</p>	$C_s = 0.709$
Equation 7.4-1 of Section 7.4 ASCE 7	<p><b>Sloped Roof Snow Load (Balanced), <math>p_s</math></b></p> <p>The sloped roof snow load, <math>p_s</math>, (psf) is calculated using the Equation 7.4-1:</p> $p_s = C_s p_f$ $p_s = (0.709)(6.05) = 4.29psf$	$p_s = 4.29 \text{ psf}$

## Project Details

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

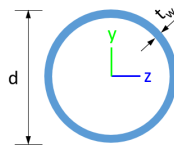


## Design Input Information

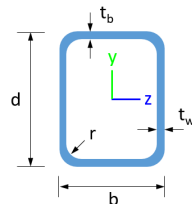
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

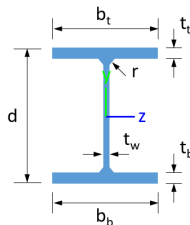
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
8	6in Pipe Sch 80	6.63	0.43				



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



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

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60

16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K <sub>z</sub> L (ft)	K <sub>y</sub> L (ft)	L <sub>b</sub> (ft)	C <sub>b</sub>	L	S	T
1	8	22.20	22.20	10.57	-	3	2	1
2	5	1.30	1.30	2.00	-	3	2	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.17,1.16,1.17,1.17,1.17,1.16,1.17,1.17,1.17,1.03,1.15,1.17,1.17,1.15,1.16,1.17,1.17,1.17	3	2	1
4	16	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.84,1.67,1.67,1.47,1.69,1.68,1.68,1.64,1.71,1.67,1.67,1.66,1.45	3	2	1
5	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.55,1.65,1.67,1.67,1.65,1.65,1.67,1.67,1.66,1.66	3	2	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.19,1.19,1.18,1.18,1.19,1.19,1.15,1.18,1.19,1.19,1.17,1.18,1.19,1.19,1.18,1.18	3	2	1
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.63,1.66,1.67,1.67,1.66,1.67,1.67,1.66,1.67	3	2	1
8	19	1.33	1.33	2.05	1.30,1.30,1.30,1.30,1.30,1.30,1.29,1.29,1.29,1.28,1.29,1.29,1.29,1.05,1.29,1.29,1.22,1.43,1.29,1.29,1.28,1.98,1.29,1.29,1.29,1.03	3	2	1
9	2	2.60	2.60	4.00	-	3	2	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.67,1.66,2.43,1.67,1.67,1.66,1.57,1.67,1.67,1.56,1.69,1.67,1.67,1.65,1.72,1.67,1.67,1.66,1.63	3	2	1
11	19	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.38,1.38,1.40,1.45,1.38,1.38,1.39,1.44,1.36,1.36,1.85,1.63,1.37,1.37,1.42,1.48,1.38,1.38,1.39,1.43	3	2	1
12	5	1.03	1.03	1.58	-	3	2	1
13	19	1.14	1.14	1.75	2.27,2.27,2.27,2.27,2.27,2.27,2.22,2.22,2.21,2.01,2.22,2.22,2.21,2.04,2.24,2.24,1.55,1.68,2.23,2.23,2.13,1.92,2.22,2.22,2.21,2.06	3	2	1
14	19	1.14	1.14	1.75	1.48,1.48	3	2	1
15	19	4.20	4.20	2.00	2.33,2.33	3	2	1
16	19	4.20	4.20	2.00	2.33,2.33	3	2	1
17	19	2.60	2.60	4.00	1.29,1.30,1.29,1.30,1.29,1.29,1.26,1.26,1.25,1.30,1.26,1.26,1.25,1.30,1.27,1.27,1.12,1.30,1.27,1.27,1.23,1.30,1.26,1.26,1.26,1.30	3	2	1
18	19	1.14	1.14	1.75	1.48,1.48	3	2	1
19	19	2.60	2.60	4.00	1.30,1.30,1.30,1.31,1.30,1.30,1.31,1.31,1.31,2.29,1.30,1.30,1.31,2.25,1.31,1.31,1.35,1.46,1.30,1.30,1.31,1.70,1.30,1.30,1.31,1.85	3	2	1
20	19	1.14	1.14	1.75	2.34,2.34,2.34,2.34,2.34,2.34,2.27,2.27,2.18,1.16,2.27,2.27,2.20,1.02,2.33,2.33,1.88,2.06,2.30,2.30,2.15,1.52,2.26,2.26,2.22,1.11	3	2	1
21	19	2.60	2.60	4.00	1.08,1.08,1.08,1.08,1.08,1.08,1.11,1.11,1.14,1.20,1.11,1.11,1.13,1.18,1.10,1.10,1.01,1.04,1.11,1.11,1.21,1.29,1.12,1.12,1.13,1.17	3	2	1
22	19	1.14	1.14	1.75	1.53,1.53,1.53,1.53,1.53,1.53,1.86,1.86,2.22,2.27,1.88,1.88,2.20,2.25,1.72,1.72,1.05,1.39,1.78,1.78,2.18,2.16,1.90,1.90,2.08,2.23	3	2	1
23	19	2.60	2.60	4.00	1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.00,1.07,1.07,1.07,1.01,1.07,1.07,1.10,1.10,1.07,1.07,1.08,1.53,1.07,1.07,1.07,1.02	3	2	1





24	133.20	126.79	32.87	6.12	40.24	43.62
25	133.20	118.19	32.87	6.12	40.24	43.62
26	133.20	126.79	32.87	6.12	40.24	43.62
27	133.20	118.19	32.87	6.12	40.24	43.62
28	133.20	126.79	32.87	6.12	40.24	43.62
29	198.33	198.26	21.95	21.95	59.50	59.50
30	198.33	197.32	21.95	21.95	59.50	59.50
101	378.22	128.90	62.23	62.23	113.47	113.47
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	126.79	32.87	6.12	40.24	43.62
114	133.20	126.79	32.87	6.12	40.24	43.62
115	133.20	69.16	16.40	6.12	40.24	43.62
116	133.20	69.16	16.87	6.12	40.24	43.62
201	378.22	128.90	62.23	62.23	113.47	113.47
202	198.33	198.26	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	102.39	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	102.39	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	126.79	32.87	6.12	40.24	43.62
214	133.20	126.79	32.87	6.12	40.24	43.62
215	133.20	69.16	16.87	6.12	40.24	43.62
216	133.20	69.16	16.56	6.12	40.24	43.62

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.060	0.586	0.062	0.028	0.005	0.637	#13	0.607	Not Required	Pass
2	0.001	0.200	0.118	0.050	0.024	0.318	#13	0.053	Not Required	Pass
3	0.003	0.390	0.014	0.038	0.002	0.391	#13	0.045	Not Required	Pass
4	0.002	0.385	0.052	0.039	0.012	0.437	#13	0.120	Not Required	Pass
5	0.002	0.241	0.022	0.039	0.006	0.248	#13	0.074	Not Required	Pass
6	0.004	0.573	0.045	0.059	0.012	0.612	#13	0.045	Not Required	Pass
7	0.005	0.355	0.073	0.057	0.018	0.368	#13	0.074	Not Required	Pass
8	0.003	0.110	0.075	0.034	0.007	0.156	#13	0.095	Not Required	Pass
9	0.003	0.065	0.046	0.003	0.002	0.111	#13	0.204	Not Required	Pass
10	0.005	0.532	0.077	0.053	0.018	0.553	#13	0.080	Not Required	Pass

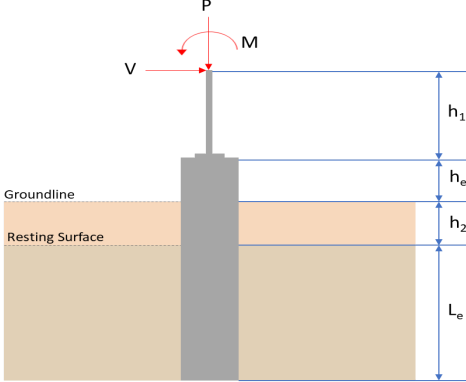
11	0.002	0.106	0.072	0.038	0.007	0.146	#13	0.095	Not Required	Pass
12	0.002	0.377	0.159	0.075	0.031	0.536	#13	0.042	Not Required	Pass
13	0.002	0.067	0.170	0.049	0.009	0.207	#16	0.081	Not Required	Pass
14	0.000	0.055	0.054	0.024	0.004	0.093	#13	Not Required	Not Required	Pass
15	0.000	0.016	0.015	0.013	0.002	0.026	#13	Not Required	Not Required	Pass
16	0.000	0.016	0.015	0.013	0.002	0.026	#13	Not Required	Not Required	Pass
17	0.001	0.080	0.038	0.016	0.003	0.101	#13	0.186	Not Required	Pass
18	0.000	0.055	0.054	0.024	0.004	0.093	#13	Not Required	Not Required	Pass
19	0.002	0.071	0.062	0.016	0.005	0.120	#13	0.186	Not Required	Pass
20	0.003	0.044	0.170	0.045	0.009	0.187	#24	0.081	Not Required	Pass
21	0.002	0.106	0.038	0.013	0.002	0.138	#13	0.186	Not Required	Pass
22	0.002	0.090	0.183	0.052	0.009	0.255	#21	0.081	Not Required	Pass
23	0.006	0.159	0.043	0.013	0.002	0.198	#13	0.186	Not Required	Pass
24	0.003	0.144	0.184	0.056	0.009	0.306	#13	0.081	Not Required	Pass
25	0.001	0.080	0.038	0.016	0.003	0.101	#13	0.186	Not Required	Pass
26	0.002	0.067	0.170	0.049	0.009	0.207	#16	0.081	Not Required	Pass
27	0.002	0.071	0.062	0.016	0.005	0.120	#13	0.186	Not Required	Pass
28	0.000	0.055	0.054	0.024	0.004	0.093	#13	Not Required	Not Required	Pass
29	0.002	0.055	0.042	0.075	0.031	0.083	#13	0.011	Not Required	Pass
30	0.002	0.377	0.159	0.075	0.031	0.536	#13	0.042	Not Required	Pass
101	0.082	0.745	0.000	0.038	0.000	0.786	#13	0.607	Not Required	Pass
102	0.002	0.398	0.190	0.086	0.036	0.589	#13	0.053	Not Required	Pass
103	0.004	0.627	0.027	0.063	0.006	0.644	#13	0.045	Not Required	Pass
104	0.004	0.660	0.078	0.066	0.016	0.703	#13	0.080	Not Required	Pass
105	0.004	0.388	0.080	0.062	0.020	0.408	#13	0.074	Not Required	Pass
106	0.004	0.627	0.027	0.063	0.006	0.644	#13	0.045	Not Required	Pass
107	0.004	0.388	0.080	0.062	0.020	0.408	#13	0.074	Not Required	Pass
108	0.003	0.058	0.083	0.044	0.007	0.121	#13	0.095	Not Required	Pass
109	0.007	0.054	0.036	0.001	0.000	0.093	#13	0.204	Not Required	Pass
110	0.004	0.660	0.078	0.066	0.016	0.703	#13	0.080	Not Required	Pass
111	0.002	0.094	0.083	0.041	0.007	0.120	#16	0.095	Not Required	Pass
112	0.002	0.398	0.190	0.086	0.036	0.589	#13	0.053	Not Required	Pass
113	0.002	0.090	0.183	0.052	0.009	0.255	#21	0.081	Not Required	Pass
114	0.003	0.144	0.184	0.056	0.009	0.305	#13	0.081	Not Required	Pass
115	0.004	0.306	0.085	0.041	0.007	0.377	#13	0.473	Not Required	Pass
116	0.005	0.273	0.088	0.044	0.007	0.340	#13	0.473	Not Required	Pass
201	0.060	0.586	0.062	0.028	0.005	0.637	#13	0.607	Not Required	Pass
202	0.002	0.055	0.042	0.075	0.031	0.083	#13	0.011	Not Required	Pass
203	0.004	0.573	0.045	0.059	0.012	0.612	#13	0.045	Not Required	Pass
204	0.005	0.532	0.077	0.053	0.018	0.553	#13	0.080	Not Required	Pass
205	0.005	0.355	0.073	0.057	0.018	0.368	#13	0.074	Not Required	Pass
206	0.003	0.390	0.014	0.038	0.002	0.391	#13	0.045	Not Required	Pass
207	0.002	0.241	0.022	0.039	0.006	0.248	#13	0.074	Not Required	Pass
208	0.000	0.016	0.015	0.013	0.002	0.026	#13	Not Required	Not Required	Pass
209	0.003	0.065	0.046	0.003	0.002	0.111	#13	0.204	Not Required	Pass
210	0.002	0.385	0.052	0.039	0.012	0.437	#13	0.120	Not Required	Pass
211	0.000	0.016	0.015	0.013	0.002	0.026	#13	Not Required	Not Required	Pass
212	0.001	0.200	0.118	0.050	0.024	0.318	#13	0.053	Not Required	Pass
213	0.000	0.055	0.054	0.024	0.004	0.093	#13	Not Required	Not Required	Pass
214	0.003	0.044	0.170	0.045	0.009	0.187	#24	0.081	Not Required	Pass
215	0.004	0.321	0.086	0.038	0.007	0.380	#13	0.473	Not Required	Pass
216	0.005	0.295	0.087	0.034	0.007	0.363	#13	0.473	Not Required	Pass

## Definitions

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$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
$F_y$	Specified minimum yield stress
$F_u$	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
$I_{yp}$	Moment of inertia about the Y axes
$I_{zp}$	Moment of inertia about the Z axes
$I_w$	Warping constant
$S_{yp}$	Plastic section modulus about the Y axis
$S_{zp}$	Plastic section modulus about the Z axis
KL	Effective length
$C_b$	Buckling modification factor (from all load combinations)
$L_b$	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
$P_n$	Nominal axial strength (tension/compression)
$M_n$	Nominal flexural strength (about Z/Y axis)
$V_n$	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
$M_z$	Design ratio in case of bending about Z axis
$M_y$	Design ratio in case of bending about Y axis
$V_y$	Design ratio in case of shear along Y axis
$V_z$	Design ratio in case of shear along Z axis
(P, $M_z$ , $M_y$ )	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.947</td> <td>7.798</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.936</td> <td>-3.249</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.321</td> <td>0.527</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>1.017</td> <td>1.670</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.594</td> <td>36.493</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.947	7.798	$V_x$ (kip)	-1.936	-3.249	$V_z$ (kip)	0.321	0.527	$M_x$ (kipft)	1.017	1.670	$M_z$ (kipft)	21.594	36.493	
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																									
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$M_x$ (kipft)	1.017	1.670																										
$M_z$ (kipft)	21.594	36.493																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.936 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.30828 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.594 \text{ kipft}) + ((-1.936 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4385 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5632 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.321 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.051115 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.017 \text{ kipft}) + ((0.321 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16194 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7798 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.5632 \text{ ft}), (2.7798 \text{ ft})]$$

$$L_{e,req} = 5.563 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.563 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92717$$

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

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

$$q = \frac{(4.947 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.30919 \text{ kip/ft}^2$$

$$q = 0.30919 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.30919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15459$$

Status: **PASS**  
Ratio: **0.150**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.30828 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.4385 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.4385 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.4385 \text{ kipft/ft})) + (4 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.132 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.4385 \text{ kipft/ft})) + (3 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.4385 \text{ kipft/ft})) + (2 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.21199 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.4385 \text{ kipft/ft})) + ((-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.8379 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.132 \text{ ft})}{2}$$

$$p_a = 0.3099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21199 \text{ kip/ft}^2)}{(0.3099 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.68406$$

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

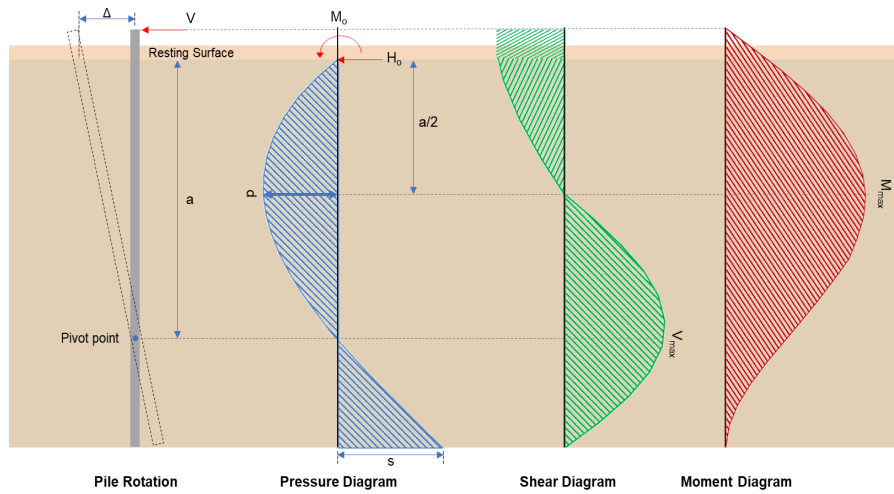
Status: **PASS**  
Ratio: **0.680**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.8379 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.931$	Status: <b>PASS</b> Ratio: <b>0.930</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.051115 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.16194 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.16194 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.051115 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.16194 \text{ kipft/ft})) + (4 \times (0.051115 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.279 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.16194 \text{ kipft/ft})) + (3 \times (0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.16194 \text{ kipft/ft})) + (2 \times (0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.046589 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.16194 \text{ kipft/ft})) + ((0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.1051 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.279 \text{ ft})}{2}$ $p_a = 0.32093 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.046589 \text{ kip/ft}^2)}{(0.32093 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.14517$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.150</b>

$$\text{Ratio} = \frac{(0.1051 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.11677$$

Status: **PASS**  
Ratio: **0.120**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.249 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.51736 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.493 \text{ kipft}) + ((-3.249 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.811 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.811 \text{ kipft/ft})}{(-0.51736 \text{ kip/ft})}$$

$$E = 11.232 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.811 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.51736 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.811 \text{ kipft/ft})) + (4 \times (-0.51736 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1313 \text{ ft}$$

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

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

$$V_{max} = ((-0.51736 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1313 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1313 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2206 \text{ kip}$$

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

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

$$M_{max} = ((-0.51736 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(11.232 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.533 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.083917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.67 \text{ kipft}) + ((0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.26592 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.26592 \text{ kipft/ft})}{(0.083917 \text{ kip/ft})}$$

$$E = 3.1689 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.26592 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.083917 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.26592 \text{ kipft/ft})) + (4 \times (0.083917 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.279 \text{ ft}$$

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

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

$$V_{max} = ((0.083917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.279 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.279 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.53716 \text{ kip}$$

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

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

$$M_{max} = ((0.083917 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(3.1689 \text{ ft})}{(6 \text{ ft})} + \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.4317 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 3 \text{ ksi}$  - 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,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.708 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.01 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-102.01 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

#### Ties:

25.7.2.2 Since longitudinal reinforcement is  $\leq$  No. 10: Use #3(0.375 in)

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min} (D, b)]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min} ((48 \text{ in}), (48 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

#### Summary:

Main reinforcement: 14 - #5 (0.625 in)

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

$$\text{Ratio} = \frac{P}{\phi P_N}$$

$$\text{Ratio} = \frac{(7.798 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0024496$$

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

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

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

$\lambda_s$  - size effect modification factor

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 7.798 \text{ kip} \rightarrow 7798 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(7798 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.83 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

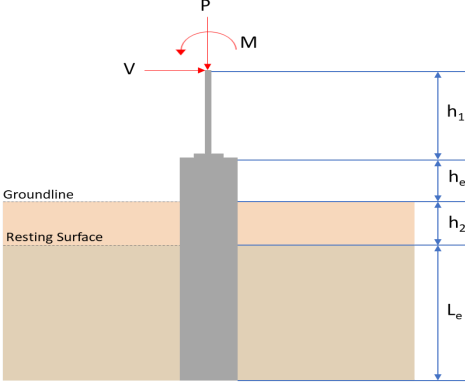
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.83 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.83 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.83 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.12 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.2206 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2206 \text{ kip})}{(118.12 \text{ kip})}$ $\text{Ratio} = 0.069593$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.53716 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.53716 \text{ kip})}{(118.12 \text{ kip})}$ $\text{Ratio} = 0.0045475$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 23.533\text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(23.533\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.086069$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 1.4317\text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.4317\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0052363$	<p>Status: <b>PASS</b>  Ratio: <b>0.010</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>4.947</td> <td>7.798</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.936</td> <td>-3.249</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.321</td> <td>-0.527</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-1.017</td> <td>-1.670</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.594</td> <td>36.494</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	4.947	7.798	$V_x$ (kip)	-1.936	-3.249	$V_z$ (kip)	-0.321	-0.527	$M_x$ (kipft)	-1.017	-1.670	$M_z$ (kipft)	21.594	36.494	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.936 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.30828 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.594 \text{ kipft}) + ((-1.936 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4385 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5632 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.321 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.051115 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.017 \text{ kipft}) + ((-0.321 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16194 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9193 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(5.5632 \text{ ft}), (1.9193 \text{ ft})]$$

$$L_{e,req} = 5.563 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(5.563 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92717$$

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

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

$$q = \frac{(4.947 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.3092 \text{ kip/ft}^2$$

$$q = 0.30919 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.30919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15459$$

Status: **PASS**  
Ratio: **0.150**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.30828 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.4385 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (3.4385 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.4385 \text{ kipft/ft})) + (4 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.132 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.4385 \text{ kipft/ft})) + (3 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.4385 \text{ kipft/ft})) + (2 \times (-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.21199 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (3.4385 \text{ kipft/ft})) + ((-0.30828 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.8379 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.132 \text{ ft})}{2}$$

$$p_a = 0.3099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21199 \text{ kip/ft}^2)}{(0.3099 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.68406$$

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

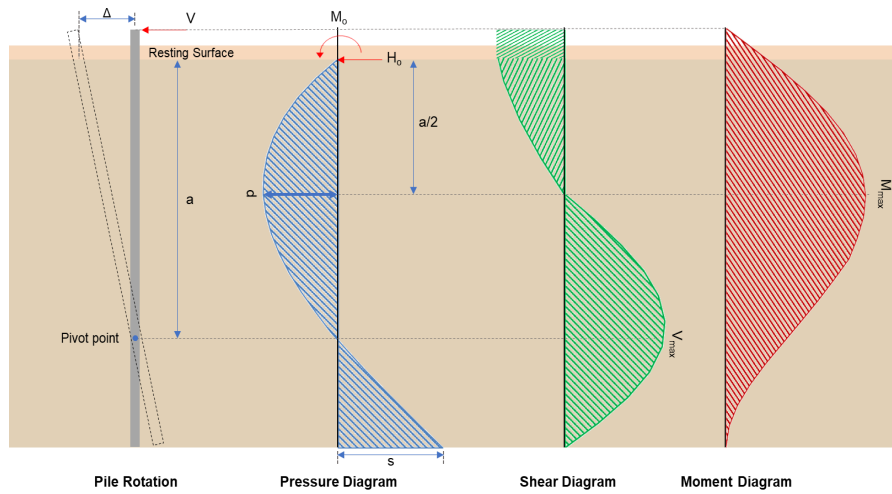
Status: **PASS**  
Ratio: **0.680**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.8379 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.931$	Status: <b>PASS</b> Ratio: <b>0.930</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.051115 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.16194 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.16194 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.051115 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.16194 \text{ kipft/ft})) + (4 \times (-0.051115 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.279 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.16194 \text{ kipft/ft})) + (3 \times (-0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.16194 \text{ kipft/ft})) + (2 \times (-0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = -0.01211 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.16194 \text{ kipft/ft})) + ((-0.051115 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.0028662 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.279 \text{ ft})}{2}$ $p_a = 0.32093 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.01211 \text{ kip/ft}^2)}{(0.32093 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.037736$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>-0.040</b>

$$\text{Ratio} = \frac{(0.0028662 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0031847$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.249 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.51736 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(36.494 \text{ kipft}) + ((-3.249 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.8111 \text{ kipft/ft}$$

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

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

$$E = \frac{(5.8111 \text{ kipft/ft})}{(-0.51736 \text{ kip/ft})}$$

$$E = 11.232 \text{ ft}$$

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

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

$$a = \frac{(4 \times (5.8111 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.51736 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.8111 \text{ kipft/ft})) + (4 \times (-0.51736 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1313 \text{ ft}$$

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

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

$$V_{max} = ((-0.51736 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1313 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1313 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2208 \text{ kip}$$

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

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

$$M_{max} = ((-0.51736 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(11.232 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (11.232 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.1313 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.534 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.083917 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(1.67 \text{ kipft}) + ((-0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.26592 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.26592 \text{ kipft/ft})}{(-0.083917 \text{ kip/ft})}$$

$$E = 3.1689 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.26592 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.26592 \text{ kipft/ft})) + (4 \times (-0.083917 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.279 \text{ ft}$$

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

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

$$V_{max} = ((-0.083917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.279 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.279 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.53716 \text{ kip}$$

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

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

$$M_{max} = ((-0.083917 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[ \left( \frac{(3.1689 \text{ ft})}{(6 \text{ ft})} + \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left( \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.1689 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left( \frac{(4.279 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.4317 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 3 \text{ ksi}$  - 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,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(7.708 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.01 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-102.01 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

#### Ties:

25.7.2.2 Since longitudinal reinforcement is  $\leq$  No. 10: Use #3(0.375 in)

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min} (D, b)]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min} ((48 \text{ in}), (48 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

#### Summary:

Main reinforcement: 14 - #5 (0.625 in)

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

$$\text{Ratio} = \frac{P}{\phi P_N}$$

$$\text{Ratio} = \frac{(7.798 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0024496$$

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

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

**Parameters:**

22.5.2.2

$b_w = 48 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3

$\lambda_s$  - size effect modification factor

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  
 $V_{c,max}$  - Max shear strength of concrete

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 7.798 \text{ kip} \rightarrow 7798 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(7798 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.83 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  
 $V_{c,b}$  - Shear strength of concrete (b)

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

$V_c$  - Governing shear strength of concrete

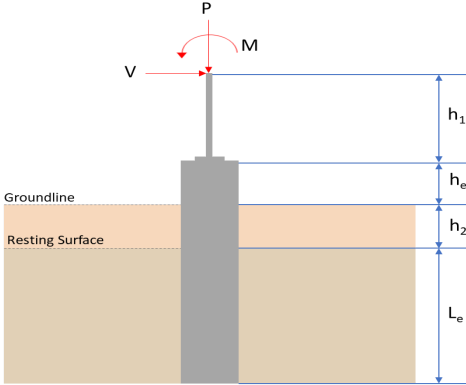
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.83 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.83 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.83 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.12 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.2208 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2208 \text{ kip})}{(118.12 \text{ kip})}$ $\text{Ratio} = 0.069595$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.53716 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.53716 \text{ kip})}{(118.12 \text{ kip})}$ $\text{Ratio} = 0.0045475$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 23.534\text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(23.534\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.086071$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 1.4317\text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.4317\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0052363$	<p>Status: <b>PASS</b>  Ratio: <b>0.010</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6.5</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>6.647</td> <td>10.546</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-2.573</td> <td>-4.275</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>27.468</td> <td>46.377</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	6.647	10.546	$V_x$ (kip)	-2.573	-4.275	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	27.468	46.377	
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$M_z$ (kipft)	27.468	46.377																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-2.573 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.40971 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(27.468 \text{ kipft}) + ((-2.573 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 4.3739 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation:  <math>L_{e,x} = 5.8961 \text{ ft}</math> - Required depth in x-direction,</p> <p><b>Considering z-direction:</b>  <math>L_{e,z} = 0 \text{ ft}</math> - Required depth in z-direction,</p> <p><b>Minimum embedded depth required:</b>  <math>L_{e,req}</math> - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.8961 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.896 \text{ ft}$ <p><math>L_e</math> - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.896 \text{ ft})}{(6.5 \text{ ft})}$ $\text{Ratio} = 0.90708$	<p>Status: <b>PASS</b>  Ratio: <b>0.910</b></p>
	<p><b>End-bearing Capacity (ASD)</b></p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(6.647 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.41544 \text{ kip/ft}^2$ <p><b>Check bearing capacity ratio:</b></p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.41544 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.20772$	<p>Status: <b>PASS</b>  Ratio: <b>0.210</b></p>
<p>Czerniak</p>	<p><b>Lateral Soil Pressure (ASD):</b></p> <p><math>L/D</math> - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.40971$  kip/ft - Lateral force per length of pile,

$M_o = 4.3739$  kipft/ft - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.3739 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.40971 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.3739 \text{ kipft/ft})) + (4 \times (-0.40971 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4897 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.3739 \text{ kipft/ft})) + (3 \times (-0.40971 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (4.3739 \text{ kipft/ft})) + (2 \times (-0.40971 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.20578 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (4.3739 \text{ kipft/ft})) + ((-0.40971 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.86409 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4897 \text{ ft})}{2}$$

$$p_a = 0.33673 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20578 \text{ kip/ft}^2)}{(0.33673 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61112$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

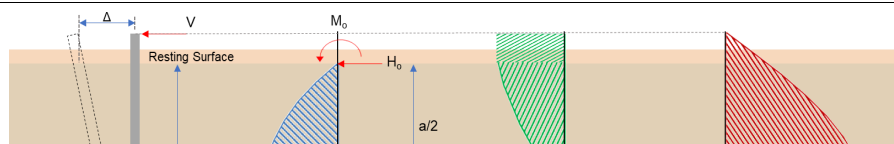
$$\text{Ratio} = \frac{s}{p_s}$$

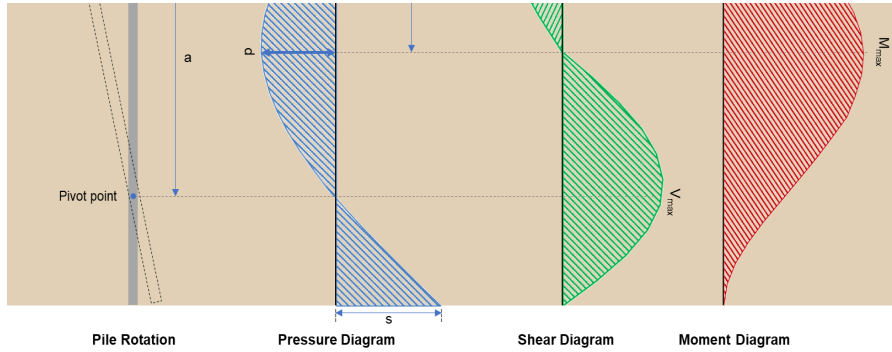
$$\text{Ratio} = \frac{(0.86409 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88625$$

Status: **PASS**  
Ratio: **0.610**

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





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-4.275 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.68073 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(46.377 \text{ kipft}) + ((-4.275 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.3849 \text{ kipft/ft}$$

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

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

$$E = \frac{(7.3849 \text{ kipft/ft})}{(-0.68073 \text{ kip/ft})}$$

$$E = 10.848 \text{ ft}$$

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

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

$$a = \frac{(4 \times (7.3849 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.68073 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.3849 \text{ kipft/ft})) + (4 \times (-0.68073 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4879 \text{ ft}$$

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

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

$$V_{max} = ((-0.68073 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (10.848 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4879 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (10.848 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4879 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.8373 \text{ kip}$$

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

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

$$M_{max} = ((-0.68073 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(10.848 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4879 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (10.848 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4879 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (10.848 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4879 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 30.365 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 3 \text{ ksi}$  - 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,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.546 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.91 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-101.91 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

$$s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

#### Ties:

25.7.2.2 Since longitudinal reinforcement is  $\leq$  No. 10: Use #3(0.375 in)

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

$$s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min} (D, b)]$$

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min} ((48 \text{ in}), (48 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

#### Summary:

Main reinforcement: **14 - #5 (0.625 in)**

**Axial Compression Strength (ACI 318-19, LRFD)**22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}] + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

$$\text{Ratio} = \frac{P}{\phi P_N}$$

$$\text{Ratio} = \frac{(10.546 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0033128$$

Status: **PASS**  
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$  - Effective width,22.5.2.2  $d$  - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3  $\lambda_s$  - size effect modification factor

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,22.5.5.1.1  $V_{c,max}$  - Max shear strength of concrete

$$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 10.546 \text{ kip} \rightarrow 10546 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(10546 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.2 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,22.5.5.1.2  $V_{c,b}$  - Shear strength of concrete (b)

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

$$V_{c,b} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

 $V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.2 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.2 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.2 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.36 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.8373 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.8373 \text{ kip})}{(118.36 \text{ kip})}$ $\text{Ratio} = 0.083113$	<p>Status: <b>PASS</b>  Ratio: <b>0.080</b></p>
<p>14.5.2.1b</p>	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = 0.85 f'_c S_m$	

$$\phi M_{n,z} = \phi S_x F_y$$

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

$$\phi M_{n,z} = 2545.9 \text{ kipft}$$

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$

$$\phi M_n = 273.42 \text{ kipft}$$

**Considering x-direction:**

$M_{max} = 30.365 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(30.365 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.11106$$

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