

# Your Project Calculations



Project Name: Alpha6b

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=Alpha6b&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/7\\_2023](https://platform.skyciv.com/structural?preload_name=Alpha6b&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=sYA1w8NeFNtHuQPoaVCjJq48xYNTyYzz0e5jkMzvxz3RKRFcdGZdjHfs8rdnV0Mt](https://platform.skyciv.com/structural-viewer?project_id=sYA1w8NeFNtHuQPoaVCjJq48xYNTyYzz0e5jkMzvxz3RKRFcdGZdjHfs8rdnV0Mt)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	2P-22.5-8TOP-XD-57-L-5Hx6W-4A45
<b>Duty Classification:</b>	XD
<b>Module Width:</b>	40.87 in
<b>Module Length:</b>	82.44in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	6
<b>Total Number of Modules:</b>	30
<b>Desired Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	5
<b>Total Array Height at Tilt:</b>	18.12 ft
<b>Total Frame Length:</b>	39.50 ft
<b>Frame Weight:</b>	2156 lbs
<b>Array Dimensions N/S:</b>	17.24 ft
<b>Array Dimensions E/W:</b>	41.72 ft
<b>Rail Length:</b>	206.85 in
<b>Rail Spacing:</b>	3.44 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

<b>Pole Size:</b>	8in Pipe Sch 40
<b>Pole Length above Grade:</b>	11.60 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	22.5 ft

## Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 8.50 ft Pile 2: 8.50 ft
<b>Foundation Volume:</b>	4.451 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.289238 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	Nevada City, CA 95959, USA
<b>Wind Speed:</b>	90 mph
<b>Snow Load:</b>	80 psf
<b>Design Uplift Pressure:</b>	0.011099 ksf
<b>Design Downforce Pressure:</b>	-0.011099 ksf
<b>Design Snow Pressure:</b>	0.017594 ksf



### Design Disclaimer

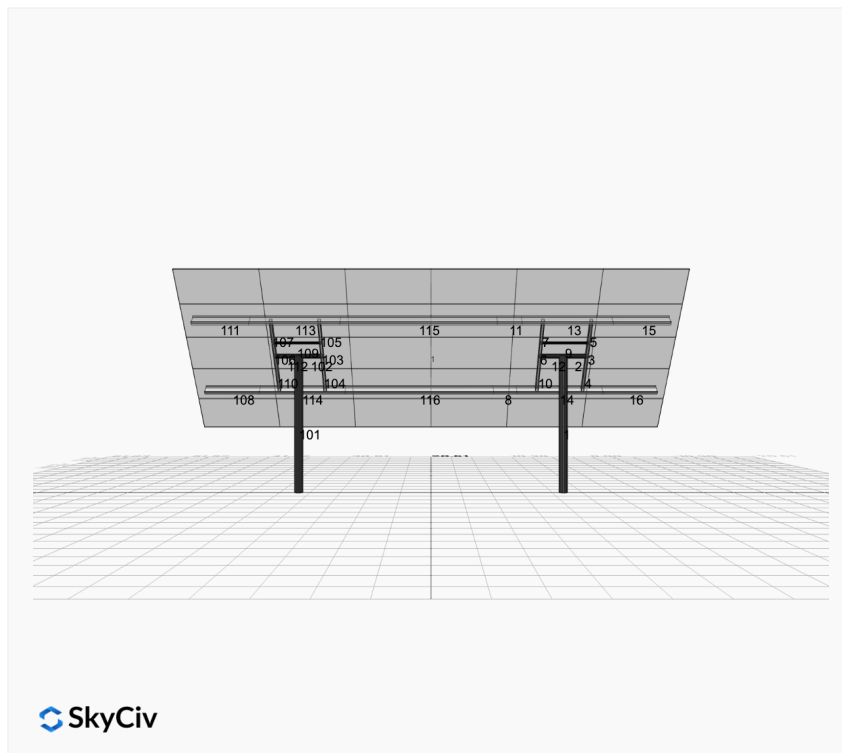
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

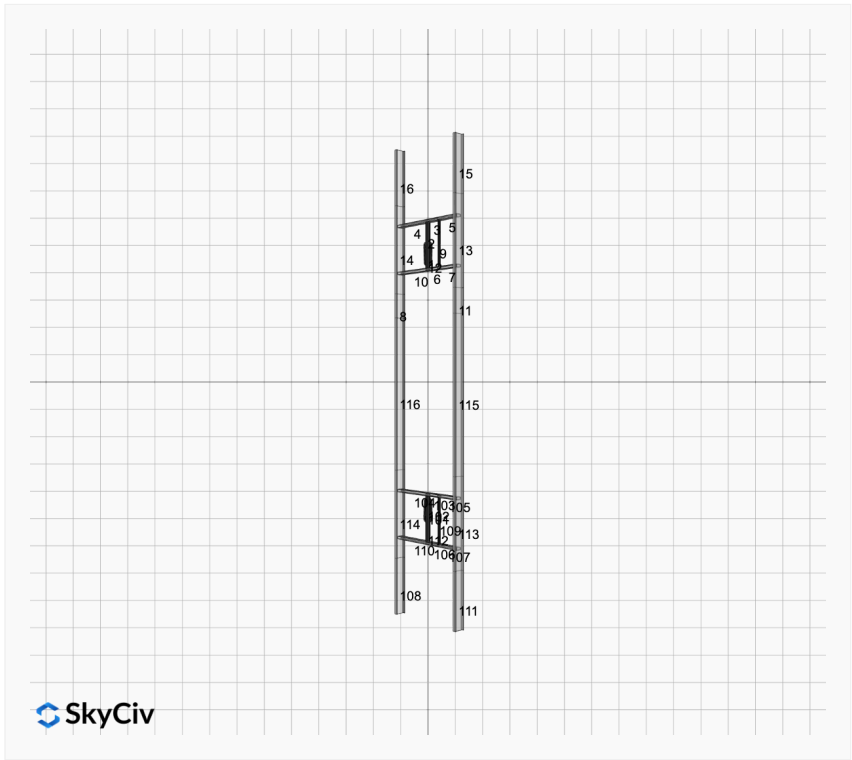
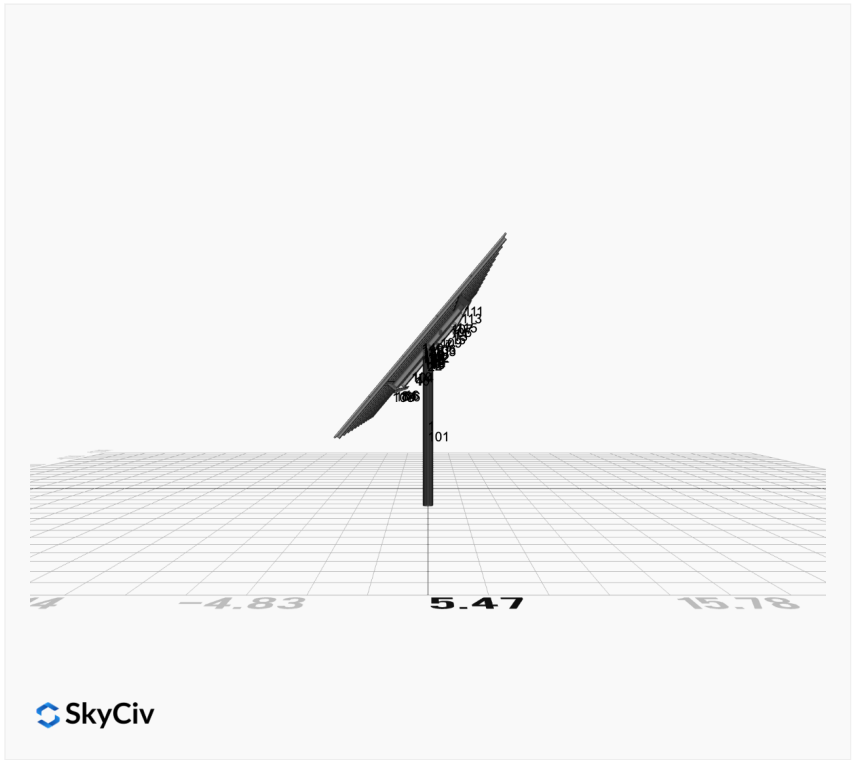
### AutoDesigner Input

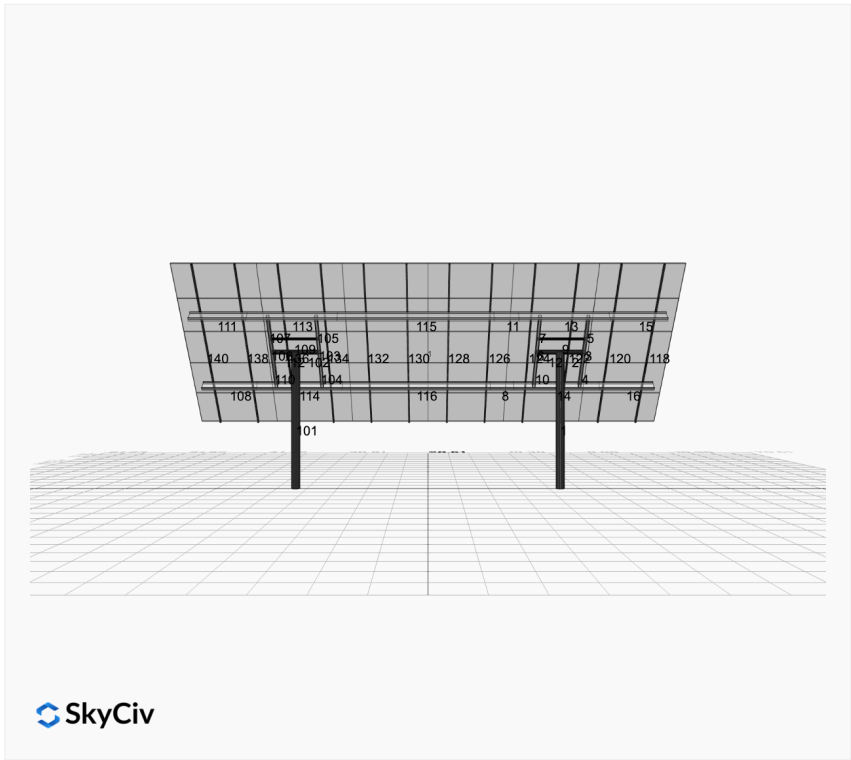
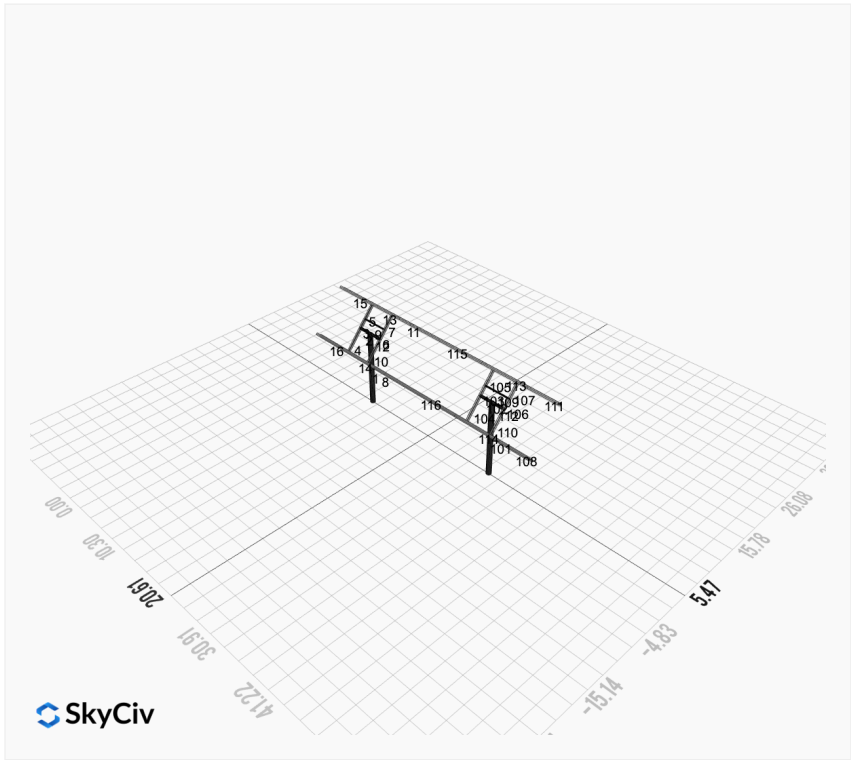
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### Design Notes:

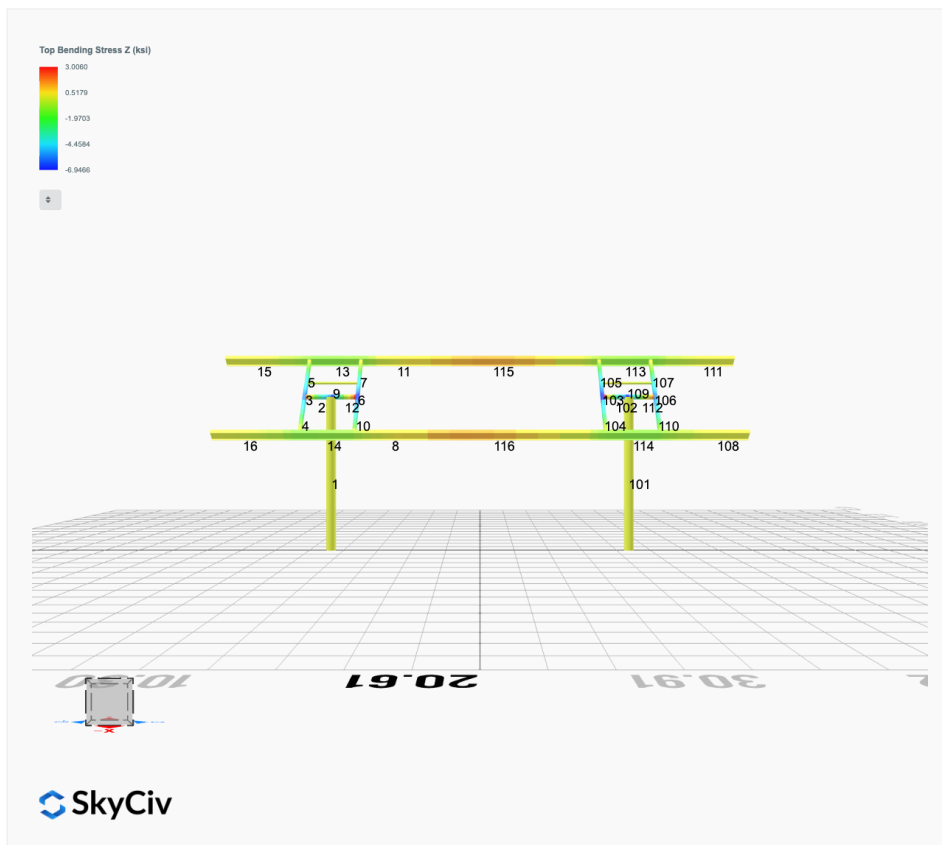
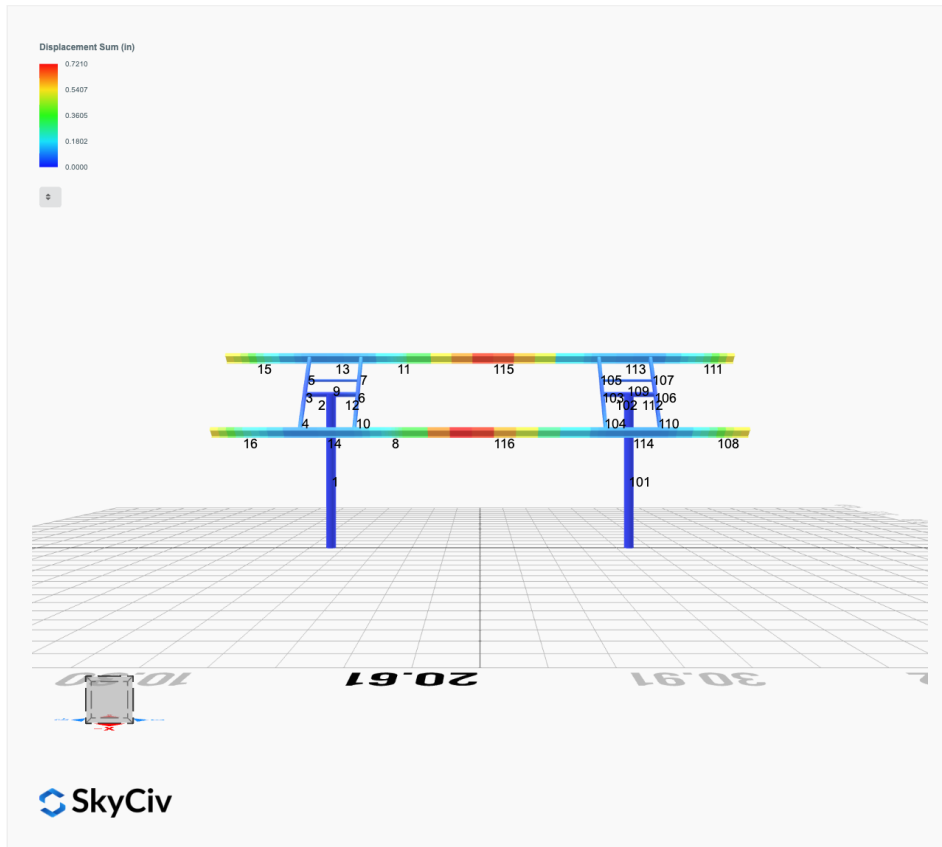
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



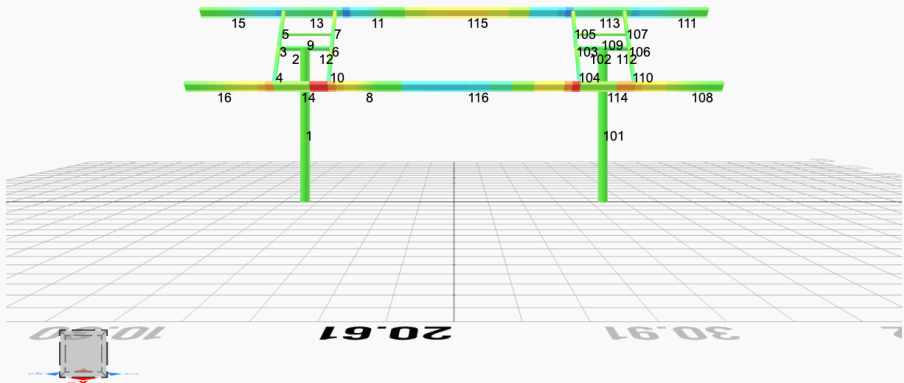




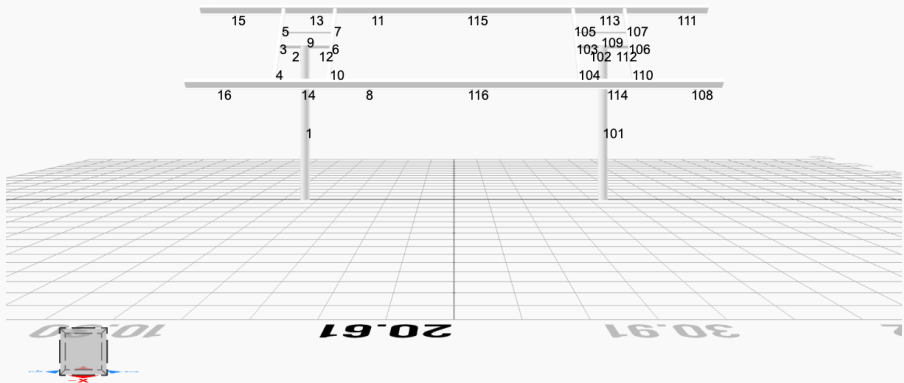
# FEM Results (Envelope Worst Case for each member)

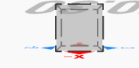
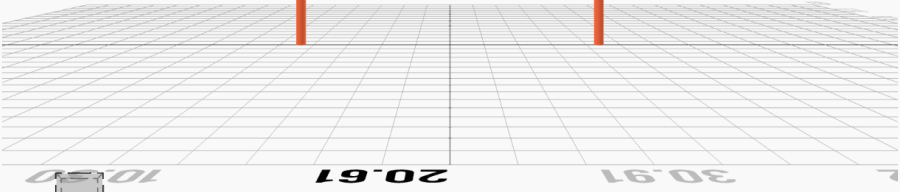
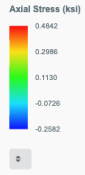


Top Bending Stress Y (ksi)



Shear Stress Y (ksi)





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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 2. D + L	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 3. D + (S or Lr or R)	0.0000	6.5465	0.1054	0.3814	-0.0478	0.0390
ULS: 3. D + (S or Lr or R)	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	5.5840	0.0883	0.3195	-0.0400	0.0352
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 5b. D + 0.7E	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	5.5840	0.0883	0.3195	-0.0400	0.0352
ULS: 8. 0.6D + 0.7E	0.0000	1.6178	0.0222	0.0801	-0.0099	0.0143
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8344	4.2356	0.0714	0.2502	-0.1699	21.4764
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8344	1.1571	0.0026	0.0172	0.1368	-21.0932
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3758	6.7384	0.1141	0.4069	-0.1550	16.1247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	5.5840	0.0883	0.3195	-0.0400	0.0352
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3758	4.4296	0.0625	0.2322	0.0751	-15.8025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	5.5840	0.0883	0.3195	-0.0400	0.0352
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3758	3.8508	0.0628	0.2210	-0.1316	16.1132
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3758	1.5419	0.0112	0.0463	0.0985	-15.8140
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.6964	0.0370	0.1336	-0.0166	0.0238
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8344	3.1571	0.0566	0.1967	-0.1633	21.4669
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.6178	0.0222	0.0801	-0.0099	0.0143
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8344	0.0786	-0.0122	-0.0362	0.1434	-21.1027
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.6178	0.0222	0.0801	-0.0099	0.0143

### 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	10.6786
Shear X	-3.0573
Shear Z	0.1832
Moment X	0.6604
Moment Y (Twist)	0.2891
Moment Z	36.4163

### 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	6.7384
Shear X	-1.8344
Shear Z	0.1141
Moment X	0.4069
Moment Y (Twist)	0.1699
Moment Z	21.4764

## 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	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 2. D + L	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 3. D + (S or Lr or R)	-0.0000	6.5465	-0.1054	-0.3816	0.0479	0.0391
ULS: 3. D + (S or Lr or R)	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	5.5840	-0.0883	-0.3196	0.0401	0.0353
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 5b. D + 0.7E	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	5.5840	-0.0883	-0.3196	0.0401	0.0353
ULS: 8. 0.6D + 0.7E	-0.0000	1.6178	-0.0222	-0.0801	0.0099	0.0143
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8344	4.2356	-0.0714	-0.2502	0.1699	21.4764
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8344	1.1571	-0.0026	-0.0172	-0.1368	-21.0932
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3758	6.7384	-0.1141	-0.4070	0.1551	16.1248
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	5.5840	-0.0883	-0.3196	0.0401	0.0353
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3758	4.4295	-0.0625	-0.2323	-0.0750	-15.8024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	5.5840	-0.0883	-0.3196	0.0401	0.0353
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3758	3.8508	-0.0628	-0.2210	0.1316	16.1133
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3758	1.5419	-0.0112	-0.0463	-0.0984	-15.8139
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.6964	-0.0370	-0.1336	0.0166	0.0238
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8344	3.1571	-0.0566	-0.1968	0.1633	21.4669
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.6178	-0.0222	-0.0801	0.0099	0.0143
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8344	0.0786	0.0122	0.0362	-0.1434	-21.1027
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.6178	-0.0222	-0.0801	0.0099	0.0143

#### 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	10.6785
Shear X	-3.0573
Shear Z	-0.1832
Moment X	-0.6613
Moment Y (Twist)	0.2892
Moment Z	36.4173

#### 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	6.7384
Shear X	-1.8344
Shear Z	-0.1141
Moment X	-0.4070
Moment Y (Twist)	0.1699
Moment Z	21.4764

## Project Details

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



## Design Input Information

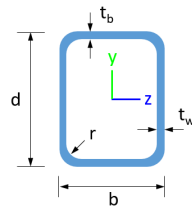
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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



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

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21





113	159.30	48.27	14.88	6.46	56.26	44.91
116	159.30	48.27	14.88	6.46	56.26	44.91

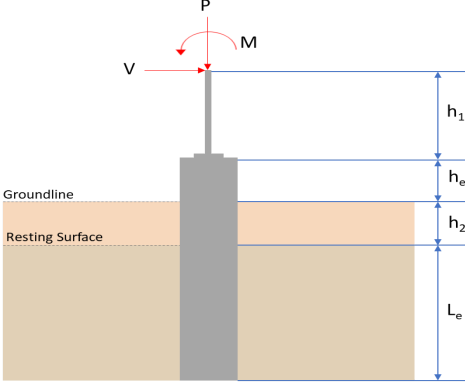
## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.058	0.437	0.018	0.027	0.002	0.464	#13	0.498	Not Required	Pass
2	0.006	0.248	0.118	0.061	0.021	0.344	#21	0.036	Not Required	Pass
3	0.010	0.340	0.056	0.034	0.008	0.377	#21	0.046	Not Required	Pass
4	0.009	0.337	0.202	0.034	0.043	0.468	#21	0.082	Not Required	Pass
5	0.009	0.211	0.204	0.034	0.054	0.268	#21	0.076	Not Required	Pass
6	0.013	0.378	0.082	0.038	0.009	0.467	#21	0.046	Not Required	Pass
7	0.013	0.234	0.273	0.037	0.071	0.307	#21	0.076	Not Required	Pass
8	0.002	0.039	0.335	0.026	0.028	0.351	#21	0.068	Not Required	Pass
9	0.027	0.028	0.058	0.002	0.002	0.074	#21	0.206	Not Required	Pass
10	0.013	0.375	0.258	0.037	0.055	0.539	#21	0.082	Not Required	Pass
11	0.004	0.039	0.345	0.026	0.028	0.362	#21	0.102	Not Required	Pass
12	0.005	0.302	0.129	0.076	0.022	0.405	#21	0.036	Not Required	Pass
13	0.010	0.134	0.728	0.032	0.035	0.836	#21	0.306	Not Required	Pass
14	0.011	0.136	0.715	0.032	0.035	0.822	#21	0.204	Not Required	Pass
15	0.000	0.046	0.294	0.016	0.018	0.340	#21	Not Required	Not Required	Pass
16	0.000	0.046	0.294	0.016	0.018	0.340	#21	Not Required	Not Required	Pass
101	0.058	0.437	0.018	0.027	0.002	0.464	#13	0.498	Not Required	Pass
102	0.005	0.302	0.129	0.076	0.022	0.405	#21	0.036	Not Required	Pass
103	0.013	0.378	0.082	0.038	0.009	0.467	#21	0.046	Not Required	Pass
104	0.013	0.375	0.259	0.037	0.055	0.539	#21	0.082	Not Required	Pass
105	0.013	0.234	0.273	0.037	0.071	0.307	#21	0.076	Not Required	Pass
106	0.010	0.340	0.056	0.034	0.008	0.377	#21	0.046	Not Required	Pass
107	0.009	0.211	0.204	0.034	0.054	0.268	#21	0.076	Not Required	Pass
108	0.000	0.046	0.294	0.016	0.018	0.340	#21	Not Required	Not Required	Pass
109	0.027	0.028	0.058	0.002	0.002	0.074	#21	0.206	Not Required	Pass
110	0.009	0.337	0.202	0.034	0.043	0.468	#21	0.082	Not Required	Pass
111	0.000	0.046	0.294	0.016	0.018	0.340	#21	Not Required	Not Required	Pass
112	0.006	0.248	0.118	0.061	0.021	0.344	#21	0.036	Not Required	Pass
113	0.010	0.134	0.727	0.032	0.035	0.836	#21	0.204	Not Required	Pass
114	0.011	0.136	0.716	0.032	0.035	0.823	#21	0.204	Not Required	Pass
115	0.010	0.308	0.392	0.026	0.028	0.705	#21	0.644	Not Required	Pass
116	0.002	0.309	0.388	0.026	0.028	0.697	#21	0.429	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	Safety factor for shear
E	Modulus of elasticity
F <sub>y</sub>	Specified minimum yield stress
F <sub>u</sub>	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I <sub>yp</sub>	Moment of inertia about the Y axes
I <sub>zp</sub>	Moment of inertia about the Z axes
I <sub>w</sub>	Warping constant
S <sub>yp</sub>	Plastic section modulus about the Y axis
S <sub>zp</sub>	Plastic section modulus about the Z axis
KL	Effective length
C <sub>b</sub>	Buckling modification factor (from all load combinations)
L <sub>b</sub>	Length between braced points

$L$	Length between brace 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: round <math>D = 36</math> in - Pile diameter <math>L = 8.5</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resisting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1079 1193 1171"> <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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>6.738</td> <td>10.679</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.834</td> <td>-3.057</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.114</td> <td>0.183</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.407</td> <td>0.660</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.476</td> <td>36.416</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	6.738	10.679	$V_x$ (kip)	-1.834	-3.057	$V_z$ (kip)	0.114	0.183	$M_x$ (kipft)	0.407	0.660	$M_z$ (kipft)	21.476	36.416	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.834 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.61133 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(21.476 \text{ kipft}) + ((-1.834 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.1587 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.6972 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.114 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.038 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.407 \text{ kipft}) + ((0.114 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.13567 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.0335 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.6972 \text{ ft}), (3.0335 \text{ ft})]$$

$$L_{e,req} = 7.697 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.697 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.90553$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(6.738 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.95323 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.95323 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.47662$$

Status: **PASS**  
Ratio: **0.480**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 2.8333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.61133 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 7.1587 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (7.1587 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (7.1587 \text{ kipft/ft})) + (4 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8977 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (7.1587 \text{ kipft/ft})) + (3 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (7.1587 \text{ kipft/ft})) + (2 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.25036 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (7.1587 \text{ kipft/ft})) + ((-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.1898 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8977 \text{ ft})}{2}$$

$$p_a = 0.44232 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25036 \text{ kip/ft}^2)}{(0.44232 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56602$$

Status: **PASS**  
Ratio: **0.570**

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1898 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9332$$

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

**Considering z-direction:**

$H_o = 0.038 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.13567 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.13567 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.038 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.13567 \text{ kipft/ft})) + (4 \times (0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1012 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.13567 \text{ kipft/ft})) + (3 \times (0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.13567 \text{ kipft/ft})) + (2 \times (0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.035383 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.13567 \text{ kipft/ft})) + ((0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.077531 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1012 \text{ ft})}{2}$$

$$p_a = 0.45759 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.035383 \text{ kip/ft}^2)}{(0.45759 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.077324$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

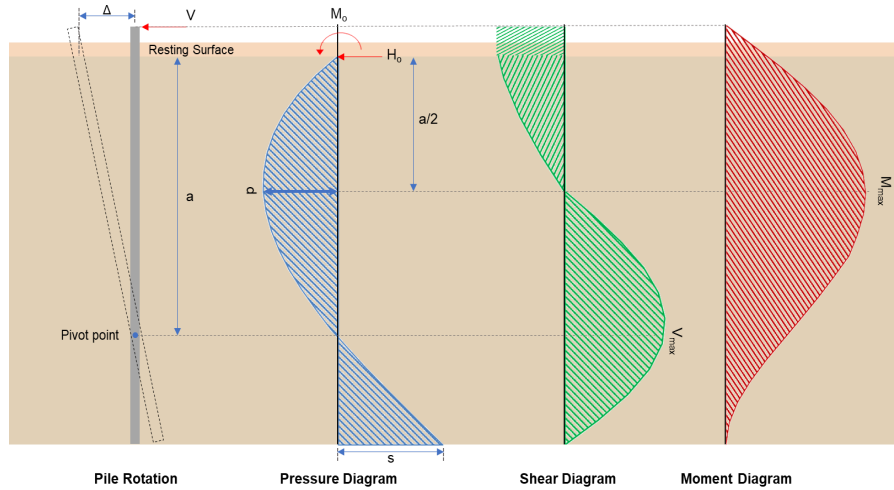
Status: **PASS**  
Ratio: **0.080**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.077531 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = 0.060808$$

Status: **PASS**  
Ratio: **0.060**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.057 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.019 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.416 \text{ kipft}) + ((-3.057 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 12.139 \text{ kipft/ft}$$

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

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

$$E = \frac{(12.139 \text{ kipft/ft})}{(-1.019 \text{ kip/ft})}$$

$$E = 11.912 \text{ ft}$$

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

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

$$a = \frac{(4 \times (12.139 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.019 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (12.139 \text{ kipft/ft})) + (4 \times (-1.019 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.895 \text{ ft}$$

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

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

$$V_{max} = ((-1.019 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.912 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.895 \text{ ft})}{(8.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (11.912 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.895 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.5967 \text{ kip}$$

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

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

$$M_{max} = ((-1.019 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(11.912 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.912 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (11.912 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 38.433 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.183 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.66 \text{ kipft}) + ((0.183 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.22 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.22 \text{ kipft/ft})}{(0.061 \text{ kip/ft})}$$

$$E = 3.6066 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.22 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.061 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.22 \text{ kipft/ft})) + (4 \times (0.061 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0995 \text{ ft}$$

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

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

$$V_{max} = ((0.061 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.6066 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0995 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (3.6066 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0995 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.25963 \text{ kip}$$

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

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

$$M_{max} = ((0.061 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(3.6066 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0995 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.6066 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0995 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (3.6066 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0995 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.965 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.85$  - Alpha factor for axial strength,  
 $A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.679 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.039 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.039 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 6$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

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 $\varnothing$ : Use #3(0.375 in)

25.7.2.1

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

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

#### Summary:

Status: **PASS**  
Ratio: **1.000**

Main reinforcement: **6 - #5 (0.625 in)**  
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

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(10.679 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0085166$$

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

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

**Parameters:**

22.5.2.2

$b_w = 36 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ .

22.5.5.1.1

$V_{c,max}$  - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.679 \text{ kip} \rightarrow 10679 \text{ lbf}$ .

22.5.5.1.1(a)

$V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(10679 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.251 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ .

22.5.5.1.2

$V_{c,b}$  - Shear strength of concrete (b)

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

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

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

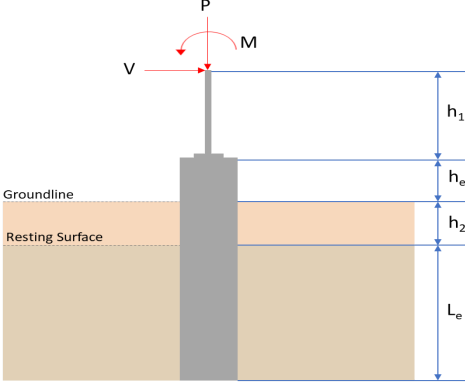
$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.251 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 76.251 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.251 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.374 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.5967 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.5967 \text{ kip})}{(74.374 \text{ kip})}$ $Ratio = 0.12903$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.25963 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.25963 \text{ kip})}{(74.374 \text{ kip})}$ $Ratio = 0.0034909$	<p>Status: <b>PASS</b> Ratio: <b>0.130</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LFRD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 38.433 \text{ kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.433 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.61963$	<p>Status: <b>PASS</b>          Ratio: <b>0.620</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.965 \text{ kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.965 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.015558$	<p>Status: <b>PASS</b>          Ratio: <b>0.020</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: round <math>D = 36</math> in - Pile diameter <math>L = 8.5</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to 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 1079 1193 1171"> <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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>6.738</td> <td>10.679</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.834</td> <td>-3.057</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.114</td> <td>-0.183</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.407</td> <td>-0.661</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>21.476</td> <td>36.417</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	6.738	10.679	$V_x$ (kip)	-1.834	-3.057	$V_z$ (kip)	-0.114	-0.183	$M_x$ (kipft)	-0.407	-0.661	$M_z$ (kipft)	21.476	36.417	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.834 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.61133 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(21.476 \text{ kipft}) + ((-1.834 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.1587 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.6972 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.114 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.038 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.407 \text{ kipft}) + ((-0.114 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.13567 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1157 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(7.6972 \text{ ft}), (2.1157 \text{ ft})]$$

$$L_{e,req} = 7.697 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(7.697 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.90553$$

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

### End-bearing Capacity (ASD)

$A$  - Pile cross-section area

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

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

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

$q$  - End-bearing pressure

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

$$q = \frac{(6.738 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.95323 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.95323 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.47662$$

Status: **PASS**  
Ratio: **0.480**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 2.8333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.61133 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 7.1587 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (7.1587 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (7.1587 \text{ kipft/ft})) + (4 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8977 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (7.1587 \text{ kipft/ft})) + (3 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (7.1587 \text{ kipft/ft})) + (2 \times (-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.25036 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (7.1587 \text{ kipft/ft})) + ((-0.61133 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.1898 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8977 \text{ ft})}{2}$$

$$p_a = 0.44232 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25036 \text{ kip/ft}^2)}{(0.44232 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56602$$

Status: **PASS**  
Ratio: **0.570**

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1898 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9332$$

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

**Considering z-direction:**

$H_o = -0.038 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.13567 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.13567 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.038 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.13567 \text{ kipft/ft})) + (4 \times (-0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.1012 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.13567 \text{ kipft/ft})) + (3 \times (-0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.13567 \text{ kipft/ft})) + (2 \times (-0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.0124 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.13567 \text{ kipft/ft})) + ((-0.038 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.0067399 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.1012 \text{ ft})}{2}$$

$$p_a = 0.45759 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0124 \text{ kip/ft}^2)}{(0.45759 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.027098$$

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

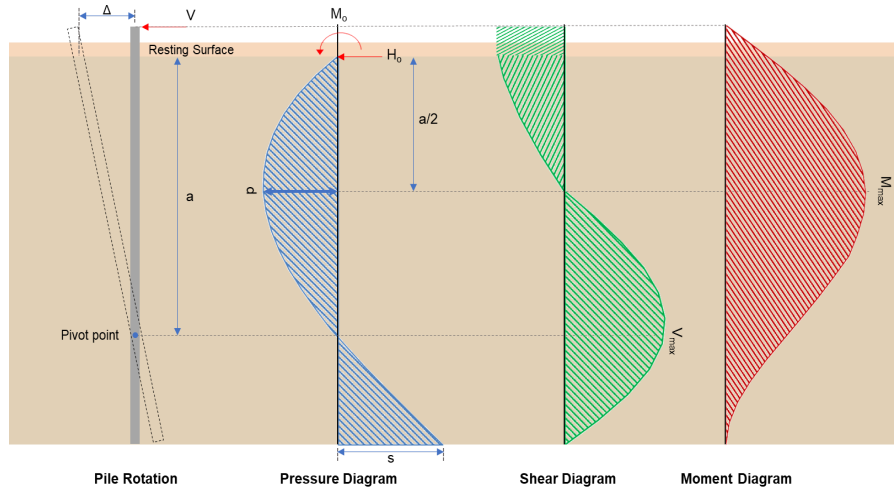
Status: **PASS**  
Ratio: **-0.030**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(-0.0067399 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$Ratio = -0.0052862$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.057 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.019 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(36.417 \text{ kipft}) + ((-3.057 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 12.139 \text{ kipft/ft}$$

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

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

$$E = \frac{(12.139 \text{ kipft/ft})}{(-1.019 \text{ kip/ft})}$$

$$E = 11.913 \text{ ft}$$

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

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

$$a = \frac{(4 \times (12.139 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-1.019 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (12.139 \text{ kipft/ft})) + (4 \times (-1.019 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.895 \text{ ft}$$

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

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

$$V_{max} = ((-1.019 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.913 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.895 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (11.913 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.895 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.597 \text{ kip}$$

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

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

$$M_{max} = ((-1.019 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(11.913 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.913 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (11.913 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(5.895 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 38.434 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.183 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.061 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.661 \text{ kipft}) + ((-0.183 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.22033 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.22033 \text{ kipft/ft})}{(-0.061 \text{ kip/ft})}$$

$$E = 3.612 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.22033 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.061 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.22033 \text{ kipft/ft})) + (4 \times (-0.061 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0993 \text{ ft}$$

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

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

$$V_{max} = ((-0.061 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (3.612 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0993 \text{ ft})}{(8.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (3.612 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0993 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.25984 \text{ kip}$$

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

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

$$M_{max} = ((-0.061 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[ \left( \frac{(3.612 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0993 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (3.612 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left( \frac{(6.0993 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (3.612 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left( \frac{(6.0993 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.96587 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.85$  - Alpha factor for axial strength,  
 $A_g = 1017.9 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(10.679 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.039 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.039 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 6$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.99533$$

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 $\varnothing$ : Use #3(0.375 in)

25.7.2.1

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

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

#### Summary:

Status: **PASS**  
Ratio: **1.000**

Main reinforcement: **6 - #5 (0.625 in)**  
Ties: **#3(0.375 in) - 10 in**

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

22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[ (0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(10.679 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0085166$$

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

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

**Parameters:**

22.5.2.2  $b_w = 36 \text{ in}$  - Effective width,  
 $d$  - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[ \sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ .

22.5.5.1.1  $V_{c,max}$  - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 10.679 \text{ kip} \rightarrow 10679 \text{ lbf}$ .

22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(10679 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.251 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ .

22.5.5.1.2  $V_{c,b}$  - Shear strength of concrete (b)

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.251 \text{ kip}), (204.04 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;"><math>V_c = 76.251 \text{ kip}</math></p> <p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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><math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.251 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.374 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 9.597 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(9.597 \text{ kip})}{(74.374 \text{ kip})}$ $Ratio = 0.12904$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.25984 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.25984 \text{ kip})}{(74.374 \text{ kip})}$ $Ratio = 0.0034937$	<p>Status: <b>PASS</b> Ratio: <b>0.130</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> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;"><math>S_m = 4580.4 \text{ in}^3</math></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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$ $\phi M_n = 62.027 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 38.434 \text{ kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(38.434 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.61964$	<p>Status: <b>PASS</b>          Ratio: <b>0.620</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.96587 \text{ kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.96587 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.015572$	<p>Status: <b>PASS</b>          Ratio: <b>0.020</b></p>