

# Your Project Calculations



Project Name: The Wilderness Reserve

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=The%20Wilderness%20Reserve&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/7\\_2024](https://platform.skyciv.com/structural?preload_name=The%20Wilderness%20Reserve&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2024)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=uhveiUnt41wR1APjxnZAX68mDvGibeR0IO1oJu3GIEcECaZiSDU7CTcK0scPKeRO](https://platform.skyciv.com/structural-viewer?project_id=uhveiUnt41wR1APjxnZAX68mDvGibeR0IO1oJu3GIEcECaZiSDU7CTcK0scPKeRO)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	2P-19.75-6TOP-SD-45-L-3Hx6W-D8DH
<b>Duty Classification:</b>	SD
<b>Module Width:</b>	40.80 in
<b>Module Length:</b>	69.40in
<b>Number of Rows:</b>	3
<b>Number of Columns:</b>	6
<b>Total Number of Modules:</b>	18
<b>Desired Tilt Angle:</b>	65
<b>Front Edge Clearance:</b>	4
<b>Total Array Height at Tilt:</b>	13.30 ft
<b>Total Frame Length:</b>	34.75 ft
<b>Frame Weight:</b>	1604 lbs
<b>Array Dimensions N/S:</b>	10.32 ft
<b>Array Dimensions E/W:</b>	35.20 ft
<b>Rail Length:</b>	123.90 in
<b>Rail Spacing:</b>	2.89 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

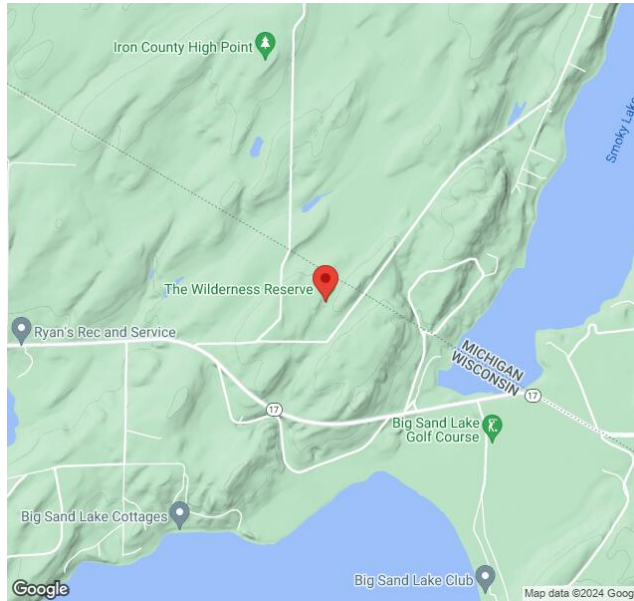
<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	8.68 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 7.00 ft Pile 2: 7.00 ft
<b>Foundation Volume:</b>	3.665 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.287206 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	680 Reserve Ln, Phelps, WI 54554, USA
<b>Wind Speed:</b>	98 mph
<b>Snow Load:</b>	60 psf
<b>Design Uplift Pressure:</b>	0.014894 ksf
<b>Design Downforce Pressure:</b>	-0.014894 ksf
<b>Design Snow Pressure:</b>	0.003299 ksf



### Design Disclaimer

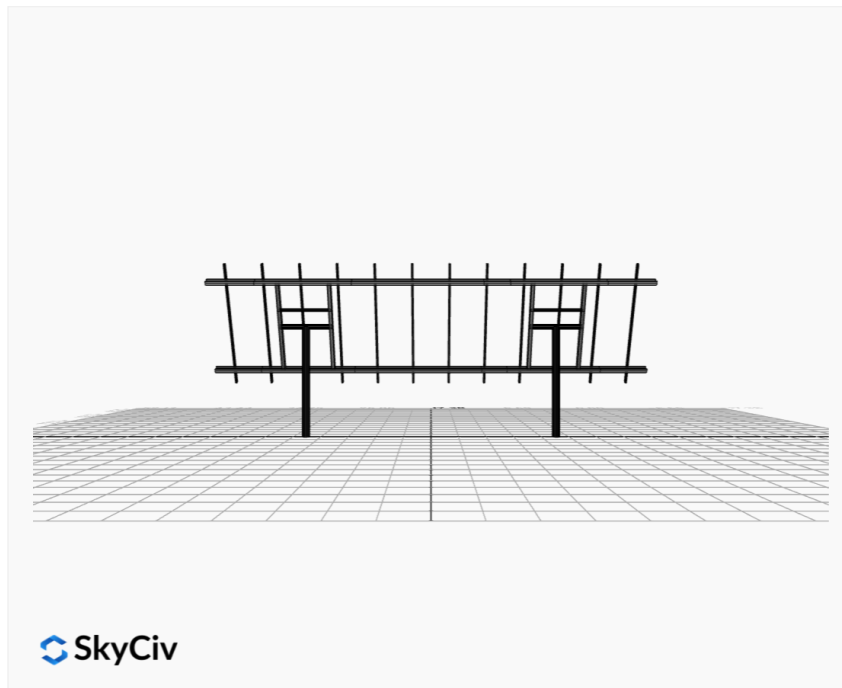
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

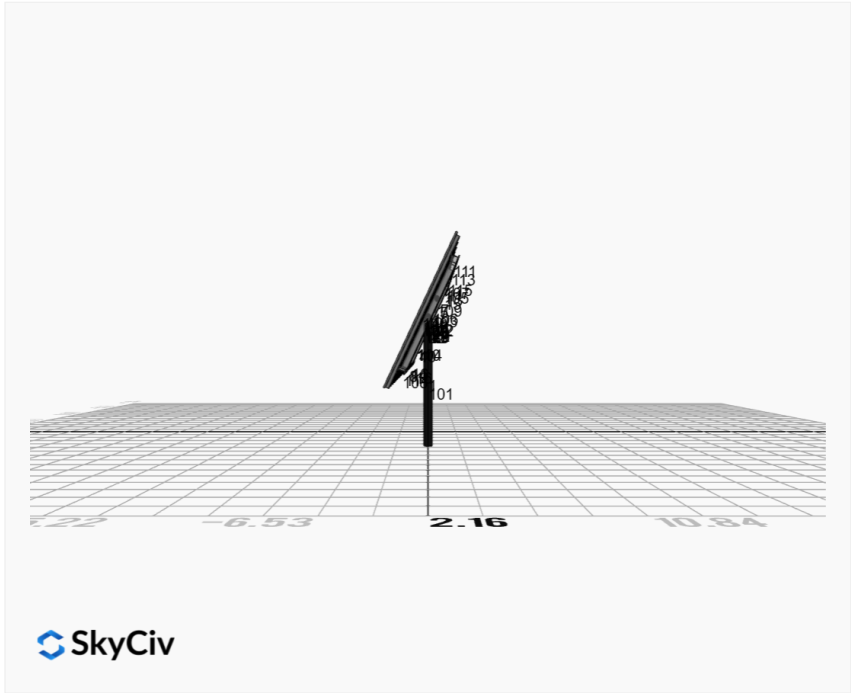
### AutoDesigner Input

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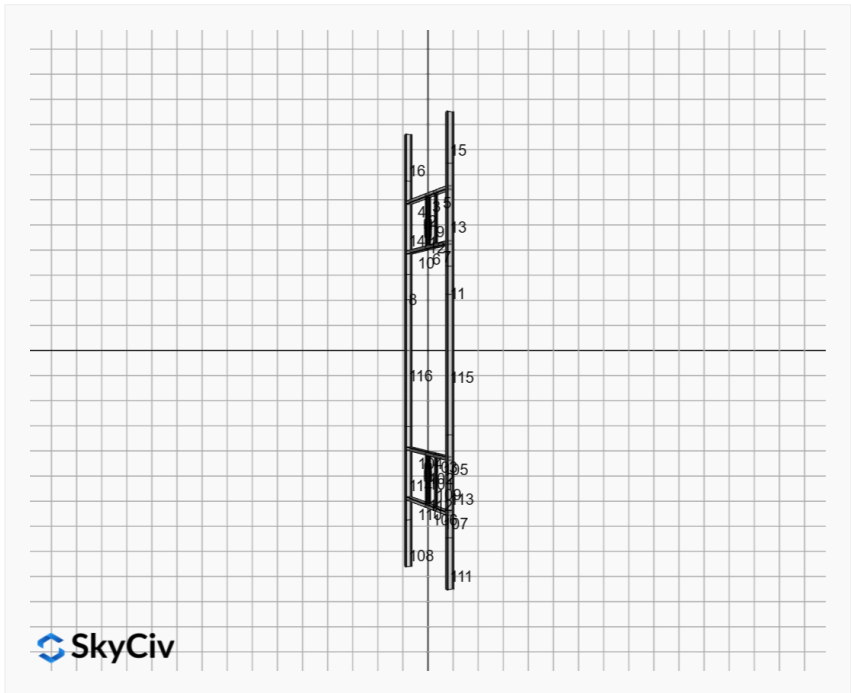
### Design Notes:

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

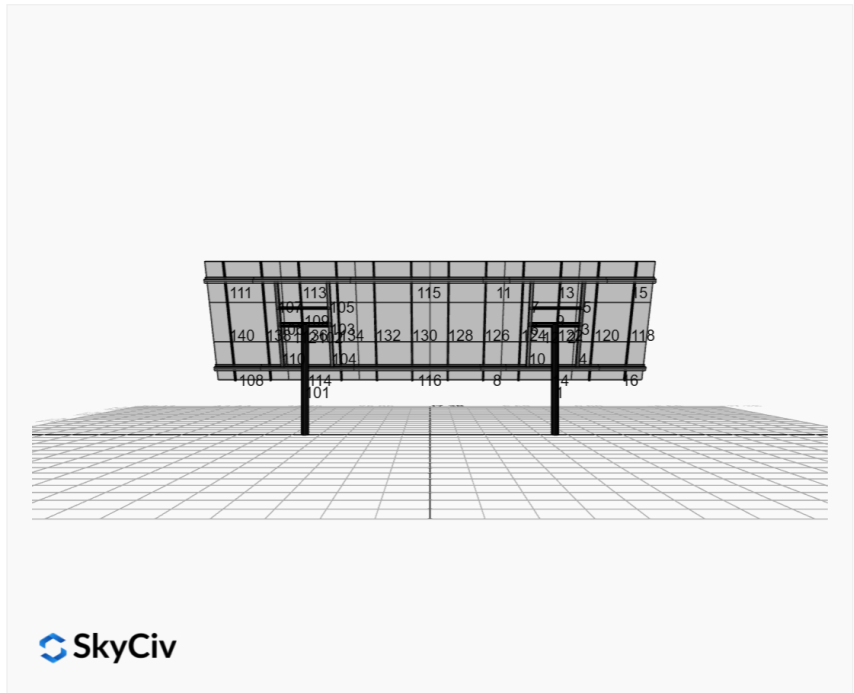
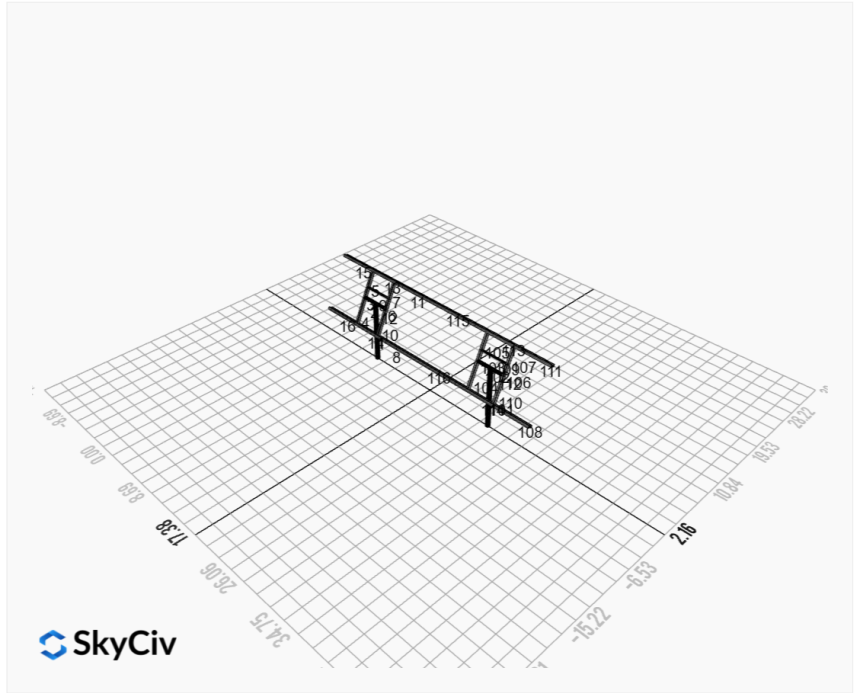




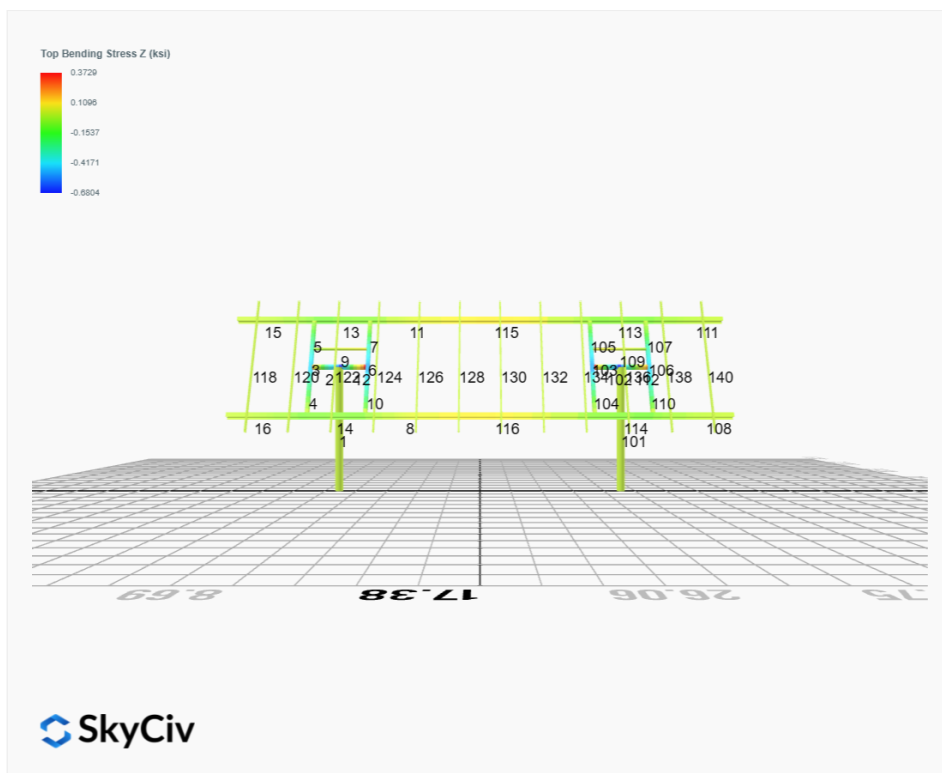
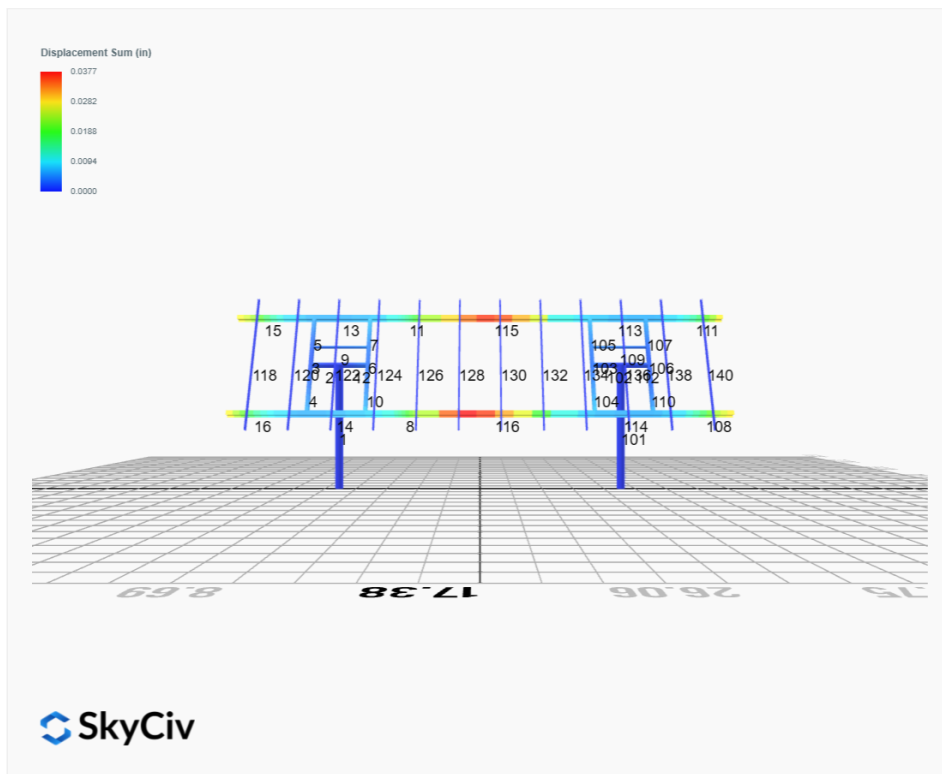
 SkyCiv

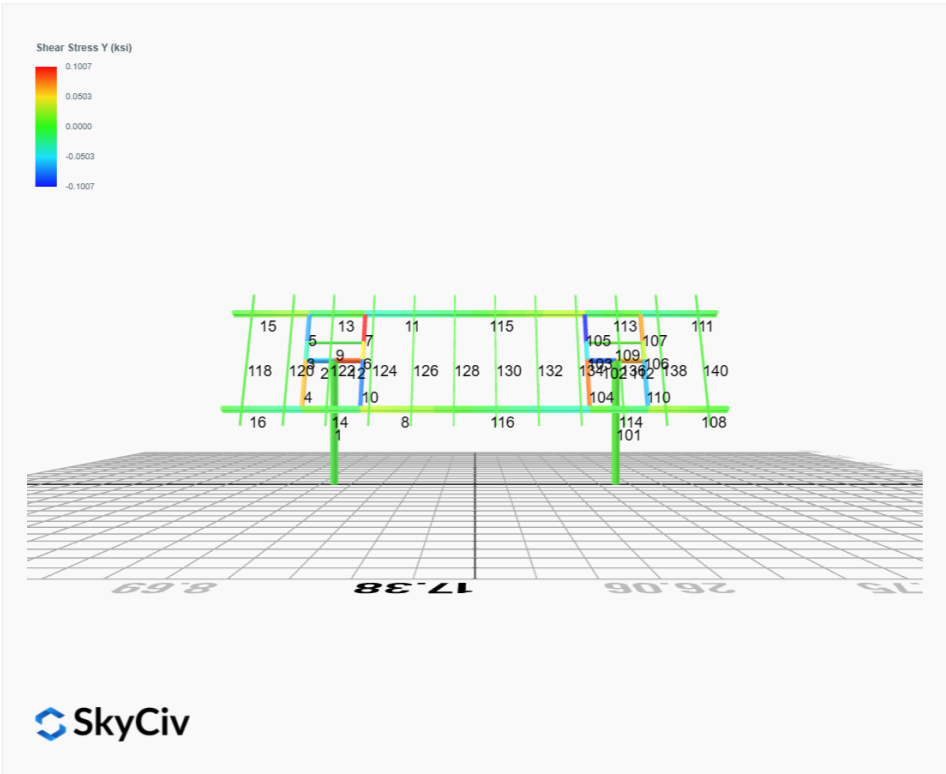
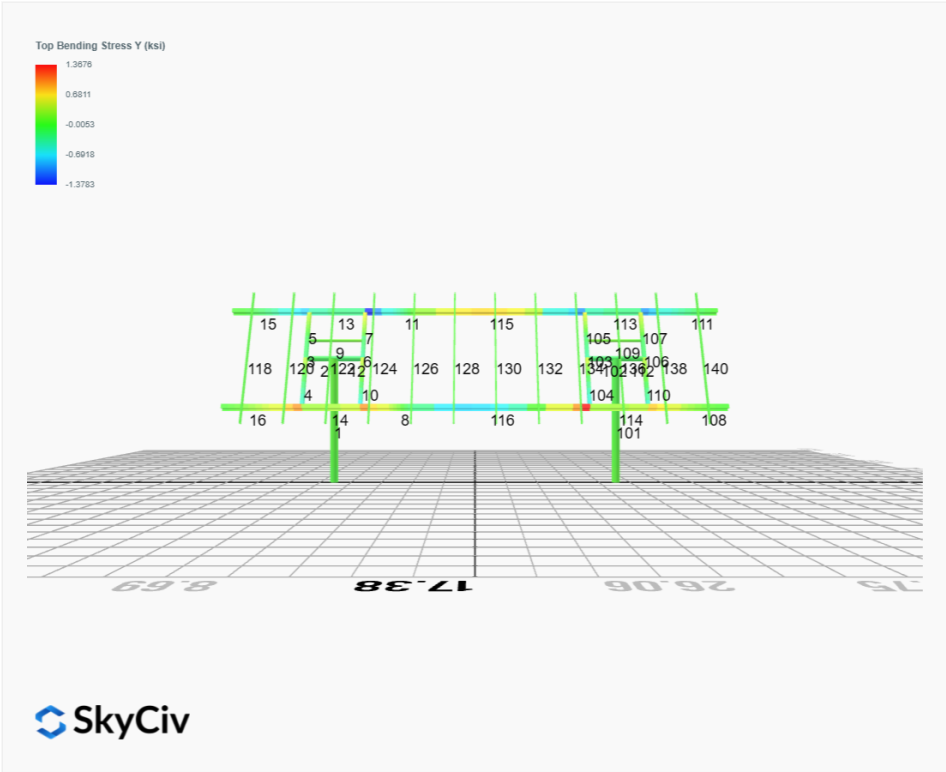


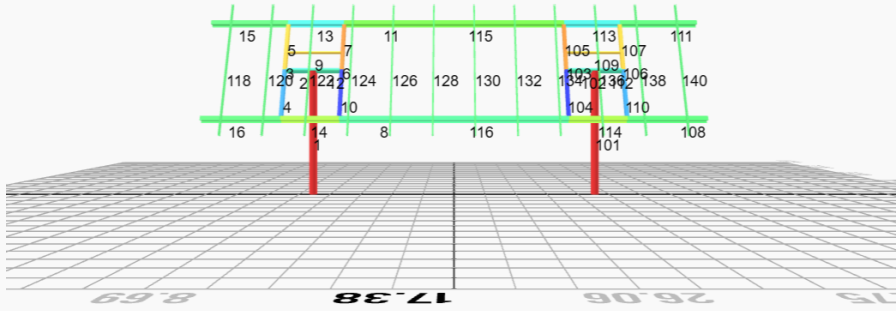
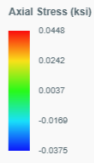
 SkyCiv



## 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	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 2. D + L	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 3. D + (S or Lr or R)	0.0000	1.7869	0.0321	0.0848	-0.0225	0.0102
ULS: 3. D + (S or Lr or R)	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7244	0.0308	0.0812	-0.0215	0.0102
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 5b. D + 0.7E	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.7244	0.0308	0.0812	-0.0215	0.0102
ULS: 8. 0.6D + 0.7E	0.0000	0.9221	0.0160	0.0422	-0.0112	0.0061
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4718	2.2231	0.0543	0.1370	-0.1769	12.8488
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4718	0.8505	-0.0009	0.0039	0.1396	-12.6991
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1039	2.2391	0.0515	0.1311	-0.1402	9.6392
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.7244	0.0308	0.0812	-0.0215	0.0102
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1039	1.2096	0.0101	0.0314	0.0972	-9.5217
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.7244	0.0308	0.0812	-0.0215	0.0102
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1039	2.0515	0.0474	0.1203	-0.1373	9.6392
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1039	1.0220	0.0060	0.0205	0.1001	-9.5218
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.5368	0.0267	0.0704	-0.0186	0.0101
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4718	1.6084	0.0436	0.1088	-0.1694	12.8448
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.9221	0.0160	0.0422	-0.0112	0.0061
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4718	0.2358	-0.0116	-0.0242	0.1471	-12.7032
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.9221	0.0160	0.0422	-0.0112	0.0061

### 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	3.1131
Shear X	-2.4530
Shear Z	0.0807
Moment X	0.2026
Moment Y (Twist)	0.2873
Moment Z	21.6177

### 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	2.2391
Shear X	-1.4718
Shear Z	0.0543
Moment X	0.1370
Moment Y (Twist)	0.1769
Moment Z	12.8488

## 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	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 2. D + L	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 3. D + (S or Lr or R)	-0.0000	1.7869	-0.0321	-0.0848	0.0225	0.0102
ULS: 3. D + (S or Lr or R)	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7244	-0.0308	-0.0812	0.0215	0.0102
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 5b. D + 0.7E	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.7244	-0.0308	-0.0812	0.0215	0.0102
ULS: 8. 0.6D + 0.7E	-0.0000	0.9221	-0.0160	-0.0422	0.0112	0.0061
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4718	2.2231	-0.0543	-0.1370	0.1769	12.8488
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4718	0.8505	0.0009	-0.0039	-0.1396	-12.6991
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1039	2.2391	-0.0515	-0.1311	0.1402	9.6392
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.7244	-0.0308	-0.0812	0.0215	0.0102
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1039	1.2096	-0.0101	-0.0314	-0.0972	-9.5217
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.7244	-0.0308	-0.0812	0.0215	0.0102
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1039	2.0515	-0.0474	-0.1203	0.1373	9.6392
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1039	1.0220	-0.0060	-0.0205	-0.1001	-9.5218
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.5368	-0.0267	-0.0704	0.0186	0.0101
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4718	1.6084	-0.0436	-0.1088	0.1694	12.8448
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.9221	-0.0160	-0.0422	0.0112	0.0061
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4718	0.2358	0.0116	0.0242	-0.1471	-12.7032
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.9221	-0.0160	-0.0422	0.0112	0.0061

#### 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	3.1131
Shear X	-2.4530
Shear Z	-0.0807
Moment X	-0.2027
Moment Y (Twist)	0.2872
Moment Z	21.6182

#### 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	2.2391
Shear X	-1.4718
Shear Z	-0.0543
Moment X	-0.1370
Moment Y (Twist)	0.1769
Moment Z	12.8488

## Project Details

Design Code: AISC 360-16 LRFD  
 Provision: LRFD  
 Country: United States

User Name: sales@mtsolar.us  
 Unit System: imperial

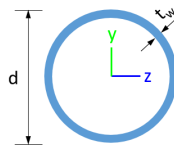


## Design Input Information

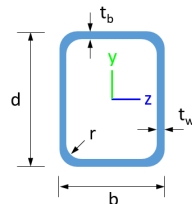
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

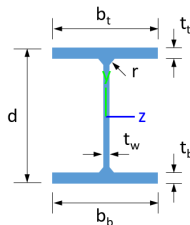
### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	$t_w$ (in)	$t_b$ (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

### 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> )
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28





115	120.00	68.63	15.22	6.45	30.09	45.74
116	120.60	68.63	15.22	6.45	30.09	45.74

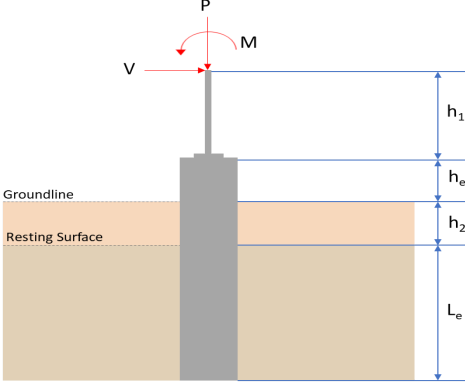
## Design Ratio

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	$\delta$	Status
1	0.025	0.511	0.012	0.033	0.001	0.528	#13	0.487	Not Required	Pass
2	0.001	0.121	0.140	0.030	0.028	0.261	#13	0.034	Not Required	Pass
3	0.006	0.278	0.046	0.028	0.004	0.318	#13	0.044	Not Required	Pass
4	0.005	0.277	0.115	0.028	0.015	0.356	#13	0.078	Not Required	Pass
5	0.005	0.172	0.111	0.028	0.016	0.193	#13	0.073	Not Required	Pass
6	0.007	0.309	0.080	0.031	0.010	0.382	#13	0.044	Not Required	Pass
7	0.007	0.191	0.155	0.031	0.023	0.225	#13	0.073	Not Required	Pass
8	0.001	0.041	0.055	0.020	0.007	0.062	#13	0.088	Not Required	Pass
9	0.005	0.019	0.038	0.002	0.001	0.046	#13	0.198	Not Required	Pass
10	0.007	0.307	0.149	0.031	0.019	0.389	#13	0.078	Not Required	Pass
11	0.001	0.041	0.056	0.020	0.007	0.061	#13	0.088	Not Required	Pass
12	0.001	0.151	0.158	0.038	0.030	0.309	#13	0.034	Not Required	Pass
13	0.002	0.082	0.149	0.025	0.008	0.191	#13	0.265	Not Required	Pass
14	0.003	0.084	0.147	0.025	0.008	0.189	#13	0.177	Not Required	Pass
15	0.000	0.029	0.053	0.012	0.004	0.075	#13	Not Required	Not Required	Pass
16	0.000	0.029	0.053	0.012	0.004	0.075	#13	Not Required	Not Required	Pass
101	0.025	0.511	0.012	0.033	0.001	0.528	#13	0.487	Not Required	Pass
102	0.001	0.151	0.158	0.038	0.030	0.309	#13	0.034	Not Required	Pass
103	0.007	0.309	0.080	0.031	0.010	0.382	#13	0.044	Not Required	Pass
104	0.007	0.307	0.149	0.031	0.019	0.389	#13	0.078	Not Required	Pass
105	0.007	0.191	0.155	0.031	0.023	0.225	#13	0.073	Not Required	Pass
106	0.006	0.278	0.046	0.028	0.004	0.318	#13	0.044	Not Required	Pass
107	0.005	0.172	0.111	0.028	0.016	0.193	#13	0.073	Not Required	Pass
108	0.000	0.029	0.053	0.012	0.004	0.075	#13	Not Required	Not Required	Pass
109	0.005	0.019	0.038	0.002	0.001	0.046	#13	0.198	Not Required	Pass
110	0.005	0.277	0.115	0.028	0.015	0.356	#13	0.078	Not Required	Pass
111	0.000	0.029	0.053	0.012	0.004	0.075	#13	Not Required	Not Required	Pass
112	0.001	0.121	0.140	0.030	0.028	0.261	#13	0.034	Not Required	Pass
113	0.002	0.082	0.149	0.025	0.008	0.191	#13	0.177	Not Required	Pass
114	0.003	0.084	0.147	0.025	0.008	0.189	#13	0.265	Not Required	Pass
115	0.002	0.115	0.086	0.020	0.007	0.189	#13	0.439	Not Required	Pass
116	0.001	0.116	0.087	0.020	0.007	0.190	#13	0.439	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>r</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 = 7</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>2.239</td> <td>3.113</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.472</td> <td>-2.453</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.054</td> <td>0.081</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.137</td> <td>0.203</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>12.849</td> <td>21.618</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)	2.239	3.113	$V_x$ (kip)	-1.472	-2.453	$V_z$ (kip)	0.054	0.081	$M_x$ (kipft)	0.137	0.203	$M_z$ (kipft)	12.849	21.618	
Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)																									
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$M_x$ (kipft)	0.137	0.203																										
$M_z$ (kipft)	12.849	21.618																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.472 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.49067 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(12.849 \text{ kipft}) + ((-1.472 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.283 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.2805 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.054 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.018 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.137 \text{ kipft}) + ((0.054 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.045667 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.1036 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.2805 \text{ ft}), (2.1036 \text{ ft})]$$

$$L_{e,req} = 6.281 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.281 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.89729$$

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

### 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

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

$$q = \frac{(2.239 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.31675 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31675 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15838$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.49067 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.283 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.283 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.283 \text{ kipft/ft})) + (4 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8699 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (4.283 \text{ kipft/ft})) + (3 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (4.283 \text{ kipft/ft})) + (2 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.18744 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (4.283 \text{ kipft/ft})) + ((-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.987 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8699 \text{ ft})}{2}$$

$$p_a = 0.36524 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18744 \text{ kip/ft}^2)}{(0.36524 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.51319$$

Status: **PASS**  
Ratio: **0.510**

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.987 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94$$

Status: **PASS**  
Ratio: **0.940**

#### Considering z-direction:

$H_o = 0.018 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.045667 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.045667 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.018 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.045667 \text{ kipft/ft})) + (4 \times (0.018 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0446 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.045667 \text{ kipft/ft})) + (3 \times (0.018 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.045667 \text{ kipft/ft})) + (2 \times (0.018 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.019427 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.045667 \text{ kipft/ft})) + ((0.018 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.041803 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0446 \text{ ft})}{2}$$

$$p_a = 0.37834 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.019427 \text{ kip/ft}^2)}{(0.37834 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.051348$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

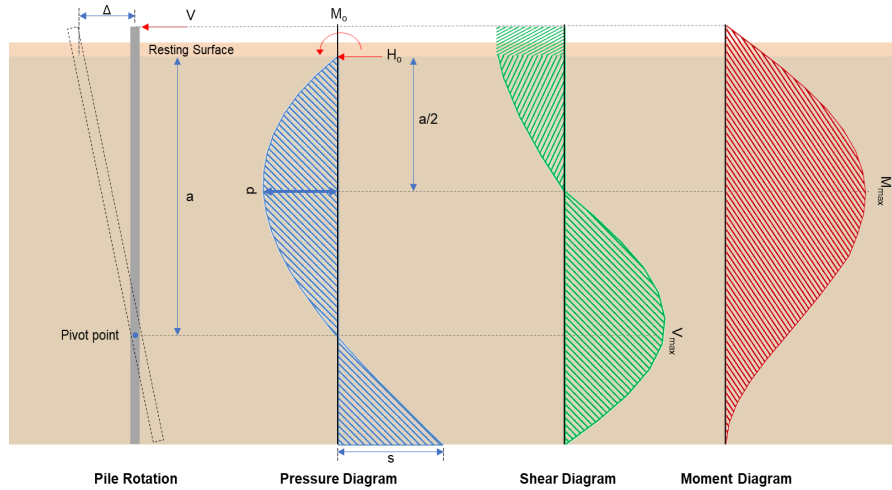
Status: **PASS**  
Ratio: **0.050**

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

$$Ratio = \frac{(0.041803 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = 0.039813$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.453 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.81767 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(21.618 \text{ kipft}) + ((-2.453 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.206 \text{ kipft/ft}$$

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

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

$$E = \frac{(7.206 \text{ kipft/ft})}{(-0.81767 \text{ kip/ft})}$$

$$E = 8.8129 \text{ ft}$$

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

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

$$a = \frac{(4 \times (7.206 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.81767 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (7.206 \text{ kipft/ft})) + (4 \times (-0.81767 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8686 \text{ ft}$$

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

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

$$V_{max} = ((-0.81767 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.8686 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.8686 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0826 \text{ kip}$$

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

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

$$M_{max} = ((-0.81767 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(8.8129 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.237 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.027 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.203 \text{ kipft}) + ((0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.067667 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.067667 \text{ kipft/ft})}{(0.027 \text{ kip/ft})}$$

$$E = 2.5062 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.067667 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.027 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.067667 \text{ kipft/ft})) + (4 \times (0.027 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0462 \text{ ft}$$

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

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

$$V_{max} = ((0.027 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(5.0462 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(5.0462 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10556 \text{ kip}$$

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

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

$$M_{max} = ((0.027 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(2.5062 \text{ ft})}{(7 \text{ ft})} + \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.31911 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(3.113 \text{ kip})}{(0.65)(0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.276 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.276 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(3.113 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0024826$$

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

**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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 3.113 \text{ kip} \rightarrow 3113 \text{ lbf}$ .

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(3113 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 74.966 \text{ kip}$$

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)

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

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

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

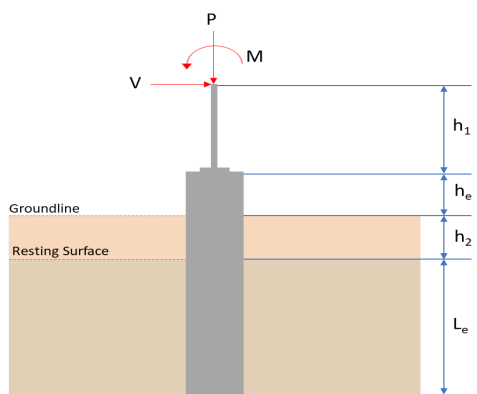
$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (74.966 \text{ kip}), (204.04 \text{ kip})]$$

<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 = 74.966 \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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((74.966 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.539 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.0826 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.0826 \text{ kip})}{(73.539 \text{ kip})}$ $Ratio = 0.096311$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.10556 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.10556 \text{ kip})}{(73.539 \text{ kip})}$ $Ratio = 0.0014355$	<p>Status: <b>PASS</b> Ratio: <b>0.100</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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <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} = 23.237 \text{ kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(23.237 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.37463$	<p>Status: <b>PASS</b>  Ratio: <b>0.370</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.31911 \text{ kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.31911 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0051447$	<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: round <math>D = 36</math> in - Pile diameter <math>L = 7</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="414 1075 1189 1176"> <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="670 1265 933 1433"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>2.239</td> <td>3.113</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.472</td> <td>-2.453</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.054</td> <td>-0.081</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.137</td> <td>-0.203</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>12.849</td> <td>21.618</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)	2.239	3.113	$V_x$ (kip)	-1.472	-2.453	$V_z$ (kip)	-0.054	-0.081	$M_x$ (kipft)	-0.137	-0.203	$M_z$ (kipft)	12.849	21.618	
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.472 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.49067 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(12.849 \text{ kipft}) + ((-1.472 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.283 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.2805 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.054 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.018 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.137 \text{ kipft}) + ((-0.054 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.045667 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4783 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.2805 \text{ ft}), (1.4783 \text{ ft})]$$

$$L_{e,req} = 6.281 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.281 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.89729$$

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

### 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

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

$$q = \frac{(2.239 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.31675 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31675 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15838$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.49067 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.283 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (4.283 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.283 \text{ kipft/ft})) + (4 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8699 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (4.283 \text{ kipft/ft})) + (3 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (4.283 \text{ kipft/ft})) + (2 \times (-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.18744 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (4.283 \text{ kipft/ft})) + ((-0.49067 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.987 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8699 \text{ ft})}{2}$$

$$p_a = 0.36524 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18744 \text{ kip/ft}^2)}{(0.36524 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.51319$$

Status: **PASS**  
Ratio: **0.510**

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.987 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94$$

Status: **PASS**  
Ratio: **0.940**

**Considering z-direction:**

$H_o = -0.018 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.045667 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (0.045667 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.018 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.045667 \text{ kipft/ft})) + (4 \times (-0.018 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0446 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.045667 \text{ kipft/ft})) + (3 \times (-0.018 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.045667 \text{ kipft/ft})) + (2 \times (-0.018 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = -0.0079763 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (0.045667 \text{ kipft/ft})) + ((-0.018 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = -0.006668 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0446 \text{ ft})}{2}$$

$$p_a = 0.37834 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0079763 \text{ kip/ft}^2)}{(0.37834 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.021082$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

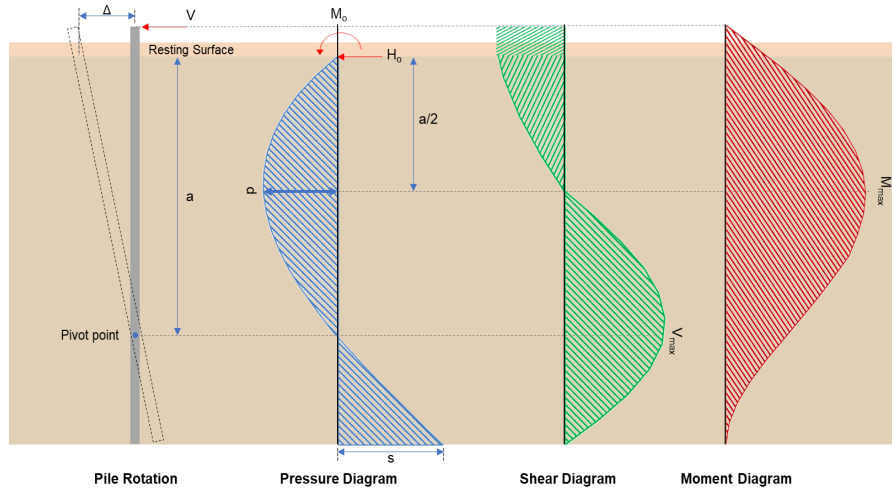
Status: **PASS**  
Ratio: **-0.020**

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

$$Ratio = \frac{(-0.006668 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$Ratio = -0.0063505$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.453 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.81767 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(21.618 \text{ kipft}) + ((-2.453 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.206 \text{ kipft/ft}$$

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

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

$$E = \frac{(7.206 \text{ kipft/ft})}{(-0.81767 \text{ kip/ft})}$$

$$E = 8.8129 \text{ ft}$$

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

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

$$a = \frac{(4 \times (7.206 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.81767 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (7.206 \text{ kipft/ft})) + (4 \times (-0.81767 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8686 \text{ ft}$$

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

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

$$V_{max} = ((-0.81767 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.8686 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.8686 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0826 \text{ kip}$$

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

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

$$M_{max} = ((-0.81767 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(8.8129 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.8129 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(4.8686 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.237 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.027 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.203 \text{ kipft}) + ((-0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.067667 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.067667 \text{ kipft/ft})}{(-0.027 \text{ kip/ft})}$$

$$E = 2.5062 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.067667 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.027 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.067667 \text{ kipft/ft})) + (4 \times (-0.027 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0462 \text{ ft}$$

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

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

$$V_{max} = ((-0.027 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(5.0462 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(5.0462 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10556 \text{ kip}$$

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

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

$$M_{max} = ((-0.027 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[ \left( \frac{(2.5062 \text{ ft})}{(7 \text{ ft})} + \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[ \left( \frac{4 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left( \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (2.5062 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left( \frac{(5.0462 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.31911 \text{ kipft}$$

### 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(3.113 \text{ kip})}{(0.65)(0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.276 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-37.276 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - 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

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

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

$$\text{Ratio} = 0.99533$$

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

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:

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

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

$$\text{Ratio} = \frac{(3.113 \text{ kip})}{(1253.9 \text{ kip})}$$

$$\text{Ratio} = 0.0024826$$

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

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

**Parameters:**

$b_w = 36 \text{ in}$  - Effective width,  
22.5.2.2  $d$  - 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  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 3.113 \text{ kip} \rightarrow 3113 \text{ lbf}$ .

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

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(3113 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 74.966 \text{ kip}$$

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

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (74.966 \text{ kip}), (204.04 \text{ kip})]$$

<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 = 74.966 \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> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p><math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((74.966 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 73.539 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 7.0826 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.0826 \text{ kip})}{(73.539 \text{ kip})}$ $\text{Ratio} = 0.096311$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.10556 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.10556 \text{ kip})}{(73.539 \text{ kip})}$ $\text{Ratio} = 0.0014355$	<p>Status: <b>PASS</b> Ratio: <b>0.100</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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ <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} = 23.237 \text{ kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(23.237 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.37463$	<p>Status: <b>PASS</b>  Ratio: <b>0.370</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.31911 \text{ kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.31911 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.0051447$	<p>Status: <b>PASS</b>  Ratio: <b>0.010</b></p>