

# Your Project Calculations



Project Name: Warren-RevA

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=Warren-RevA&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/6\\_2023](https://platform.skyciv.com/structural?preload_name=Warren-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=hyrjxKV85KS0MG5sRq0wHZes5eLjGpCUA0EUasSONXHUHLS5NHdmYXH6BWrpmxS](https://platform.skyciv.com/structural-viewer?project_id=hyrjxKV85KS0MG5sRq0wHZes5eLjGpCUA0EUasSONXHUHLS5NHdmYXH6BWrpmxS)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	2P-22.5-6TOP-SD-72-L-4Hx6W-010A
<b>Duty Classification:</b>	SD
<b>Module Width:</b>	41.30 in
<b>Module Length:</b>	83.90in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	6
<b>Total Number of Modules:</b>	24
<b>Desired Tilt Angle:</b>	30
<b>Front Edge Clearance:</b>	4
<b>Total Array Height at Tilt:</b>	10.92 ft
<b>Total Frame Length:</b>	42.00 ft
<b>Frame Weight:</b>	1347 lbs
<b>Array Dimensions N/S:</b>	13.93 ft
<b>Array Dimensions E/W:</b>	42.45 ft
<b>Rail Length:</b>	167.20 in
<b>Rail Spacing:</b>	3.54 ft
<b>Rail Check:</b>	PASS (26% utilized)

## Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	7.48 ft
<b>Number of Poles:</b>	2
<b>Pole Spacing:</b>	22.5 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.00 ft Pile 2: 5.00 ft
<b>Foundation Volume:</b>	5.926 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.227389 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	13690 Barro Ct, La Grange, CA 95329, USA
<b>Wind Speed:</b>	90 mph
<b>Snow Load:</b>	20 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.008797 ksf



### Design Disclaimer

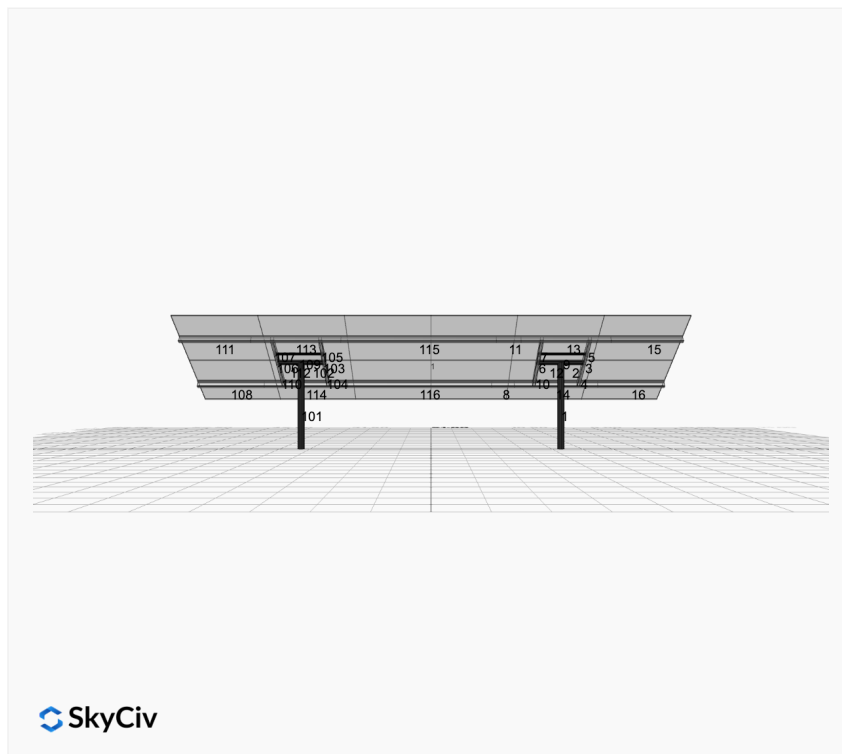
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

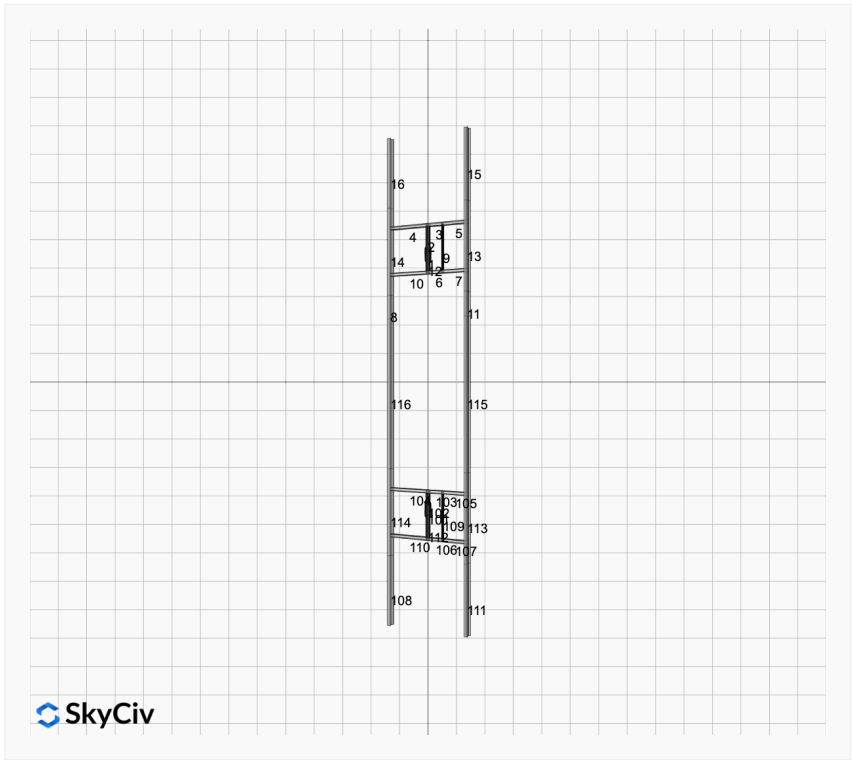
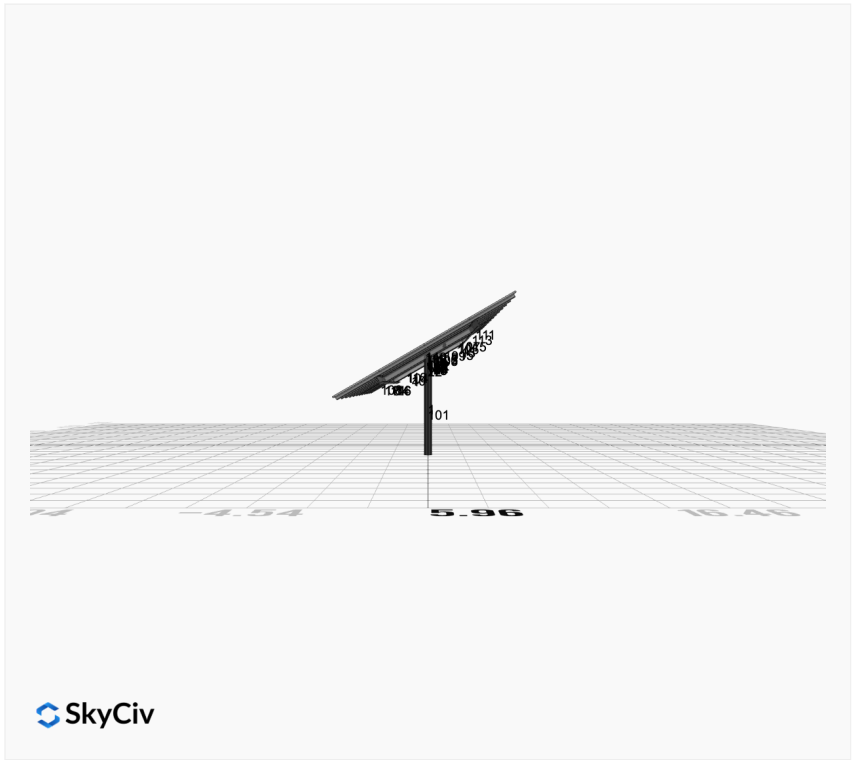
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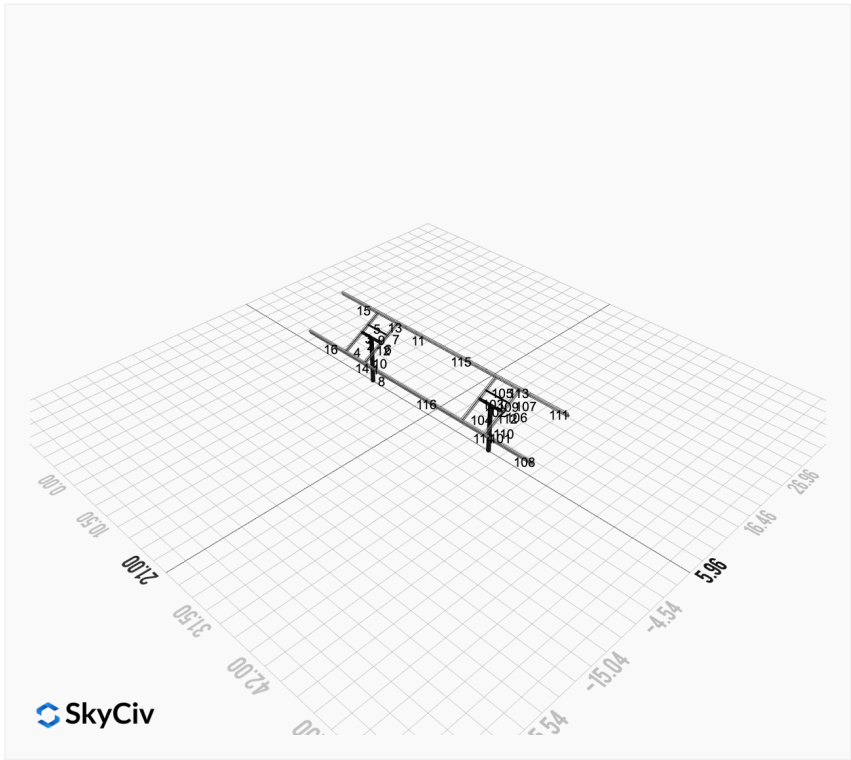
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  "soil_type": "sand",
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### Design Notes:

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







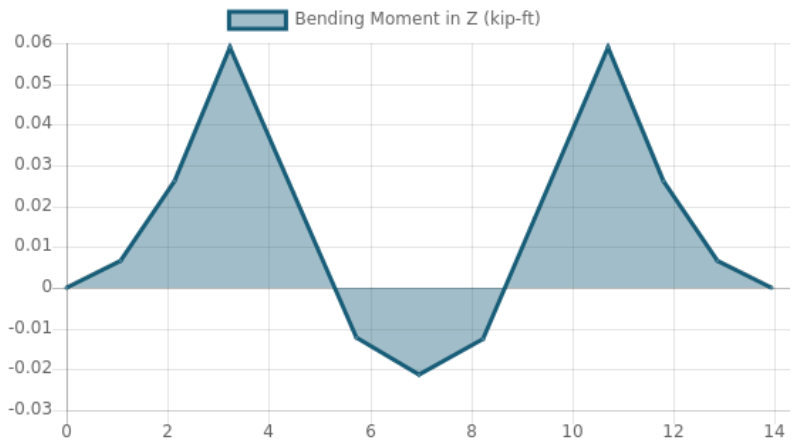
**Rail Design Check**

**Rail Length:** 13.933333333333332 ft  
**Additional Restraints Required:** None  
**Tributary Width:** 3.5375 ft  
**Material:** Aluminium  
**Density:** 169 lb/ft<sup>3</sup>  
**Elasticity Modulus:** 10000 ksi  
**Fy:** 34.5 ksi  
**Fu:** 37 ksi  
**Snow (X):** 0.0270 kip/ft  
**Snow (Y):** -0.0156 kip/ft  
**Wind uplift Case A:** 0.0528 kip/ft  
**Wind uplift Case A:** 0.0528 kip/ft  
**Wind uplift Case B (X):** 0.0000 kip/ft  
**Wind uplift Case B (Y):** 0.0733 kip/ft

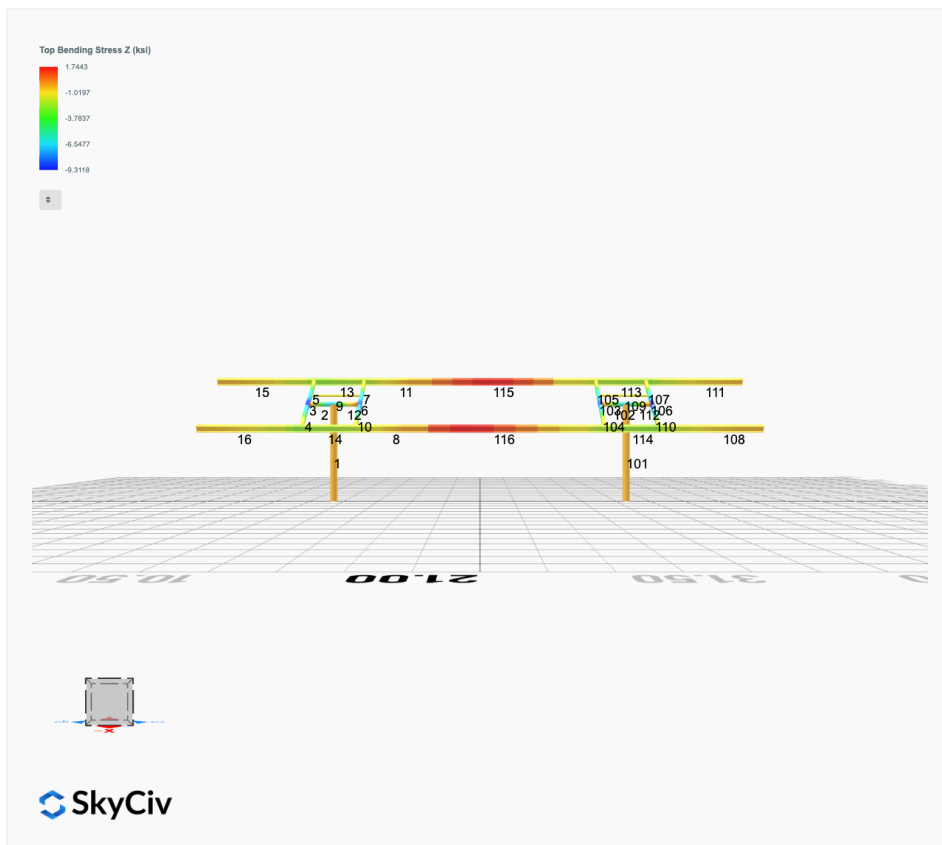
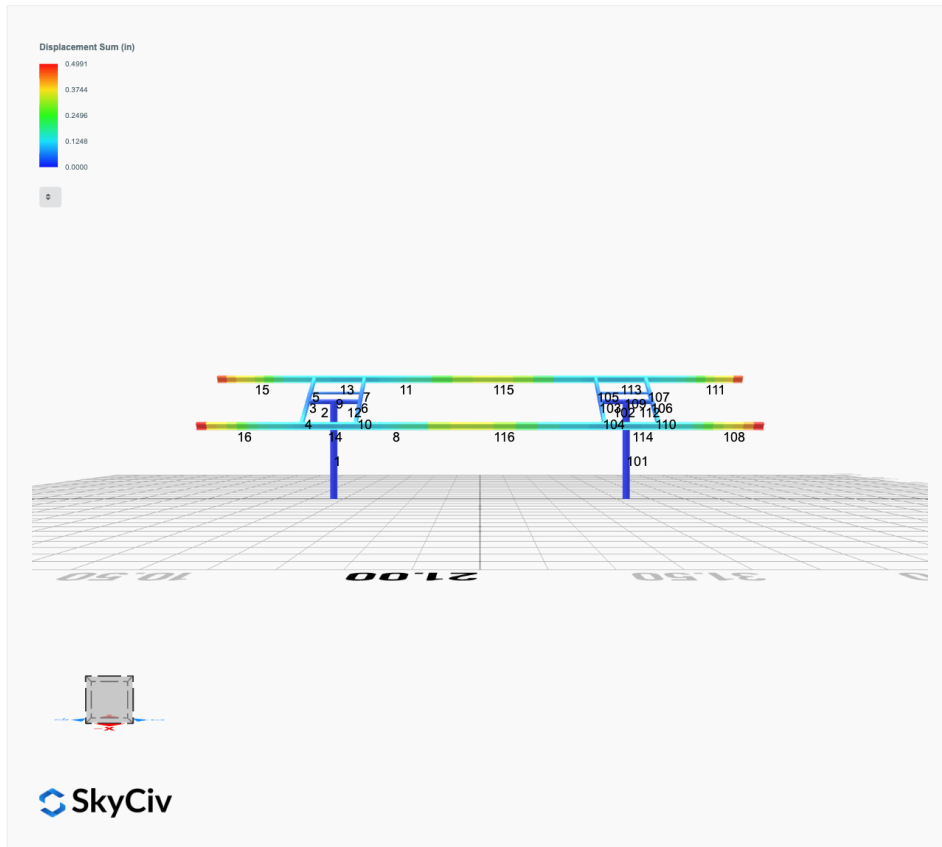


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	8.96560049	0.260	PASS
Material Yield	34.5	8.96560049	0.260	PASS
Material Strength	37	8.96560049	0.242	PASS

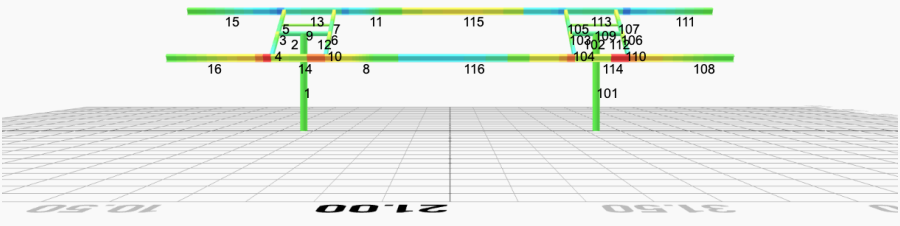
**Member 1, ULS: 1. 1.4D**



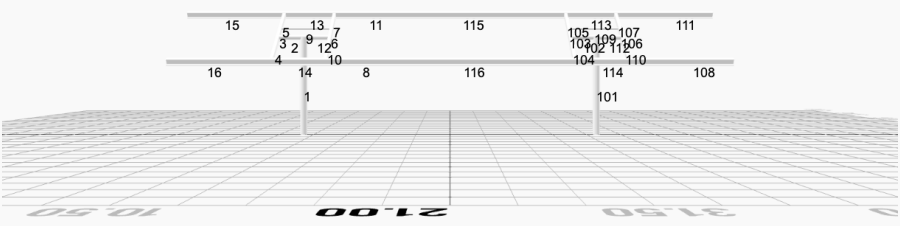
# FEM Results (Envelope Worst Case for each member)

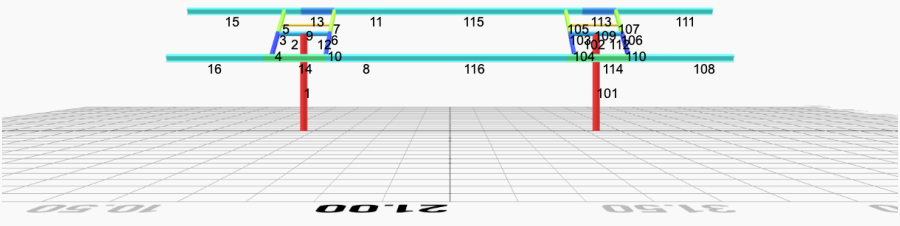
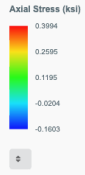


Top Bending Stress Y (ksi)



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	2.0363	-0.0246	-0.0449	0.0492	0.0212
ULS: 2. D + L	-0.0000	2.0363	-0.0246	-0.0449	0.0492	0.0212
ULS: 3. D + (S or Lr or R)	-0.0000	4.2654	-0.0559	-0.1018	0.1118	0.0291
ULS: 3. D + (S or Lr or R)	-0.0000	2.0363	-0.0246	-0.0449	0.0492	0.0212
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.7081	-0.0481	-0.0876	0.0961	0.0271
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.0363	-0.0246	-0.0449	0.0492	0.0212
ULS: 5b. D + 0.7E	-0.0000	2.0363	-0.0246	-0.0449	0.0492	0.0212
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.7081	-0.0481	-0.0876	0.0961	0.0271
ULS: 8. 0.6D + 0.7E	-0.0000	1.2218	-0.0148	-0.0269	0.0295	0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5444	4.7113	-0.0782	-0.1458	0.1308	11.7514
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5444	4.7113	-0.0782	-0.1458	0.1308	11.7514
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3238	-0.2566	0.0210	0.0409	-0.0204	-9.7623
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1032	0.1255	0.0135	0.0269	-0.0091	-13.5787
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1583	5.7144	-0.0883	-0.1632	0.1573	8.8247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1583	5.7144	-0.0883	-0.1632	0.1573	8.8247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9928	1.9885	-0.0139	-0.0232	0.0439	-7.3105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8274	2.2751	-0.0194	-0.0337	0.0524	-10.1728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1583	4.0425	-0.0648	-0.1206	0.1104	8.8188
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1583	4.0425	-0.0648	-0.1206	0.1104	8.8188
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9928	0.3166	0.0096	0.0195	-0.0030	-7.3164
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8274	0.6032	0.0040	0.0090	0.0055	-10.1787
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5444	3.8968	-0.0683	-0.1278	0.1111	11.7429
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5444	3.8968	-0.0683	-0.1278	0.1111	11.7429
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3238	-1.0711	0.0308	0.0589	-0.0401	-9.7708
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1032	-0.6890	0.0234	0.0449	-0.0288	-13.5872

### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.2393
Shear X	-2.5740
Shear Z	-0.1350
Moment X	-0.2514
Moment Y (Twist)	0.2273
Moment Z	22.9696

### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.7144
Shear X	-1.5444
Shear Z	-0.0883
Moment X	-0.1632
Moment Y (Twist)	0.1573
Moment Z	13.5872

## 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	2.0363	0.0246	0.0449	-0.0492	0.0212
ULS: 2. D + L	0.0000	2.0363	0.0246	0.0449	-0.0492	0.0212
ULS: 3. D + (S or Lr or R)	0.0000	4.2654	0.0559	0.1018	-0.1118	0.0291
ULS: 3. D + (S or Lr or R)	0.0000	2.0363	0.0246	0.0449	-0.0492	0.0212
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.7081	0.0481	0.0876	-0.0961	0.0271
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.0363	0.0246	0.0449	-0.0492	0.0212
ULS: 5b. D + 0.7E	0.0000	2.0363	0.0246	0.0449	-0.0492	0.0212

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.7081	0.0481	0.0876	-0.0961	0.0271
ULS: 8. 0.6D + 0.7E	0.0000	1.2218	0.0148	0.0269	-0.0295	0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5444	4.7113	0.0782	0.1458	-0.1308	11.7514
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.5444	4.7113	0.0782	0.1458	-0.1308	11.7514
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.3238	-0.2566	-0.0210	-0.0409	0.0204	-9.7623
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.1032	0.1255	-0.0135	-0.0269	0.0091	-13.5787
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1583	5.7144	0.0883	0.1632	-0.1573	8.8247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1583	5.7144	0.0883	0.1632	-0.1573	8.8247
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9928	1.9885	0.0139	0.0232	-0.0439	-7.3105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8274	2.2751	0.0194	0.0337	-0.0524	-10.1728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1583	4.0425	0.0648	0.1206	-0.1104	8.8188
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.1583	4.0425	0.0648	0.1206	-0.1104	8.8188
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.9928	0.3166	-0.0096	-0.0195	0.0030	-7.3164
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.8274	0.6032	-0.0040	-0.0090	-0.0055	-10.1787
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5444	3.8968	0.0683	0.1278	-0.1111	11.7429
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.5444	3.8968	0.0683	0.1278	-0.1111	11.7429
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.3238	-1.0711	-0.0308	-0.0589	0.0401	-9.7708
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.1032	-0.6890	-0.0234	-0.0449	0.0288	-13.5872

#### Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.2394
Shear X	-2.5740
Shear Z	0.1350
Moment X	0.2516
Moment Y (Twist)	0.2274
Moment Z	22.9701

#### Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.7144
Shear X	-1.5444
Shear Z	0.0883
Moment X	0.1632
Moment Y (Twist)	0.1573
Moment Z	13.5872

## Project Details

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

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

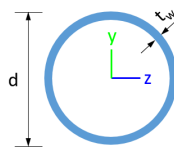


## Design Input Information

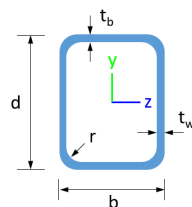
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	$F_y$ (ksi)	$F_u$ (ksi)
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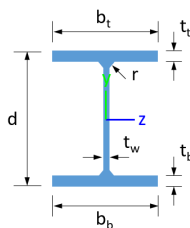
### Section Dimensions



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	46.00	11.72	6.45	30.09	45.74
116	120.60	48.60	11.72	6.45	30.09	45.74

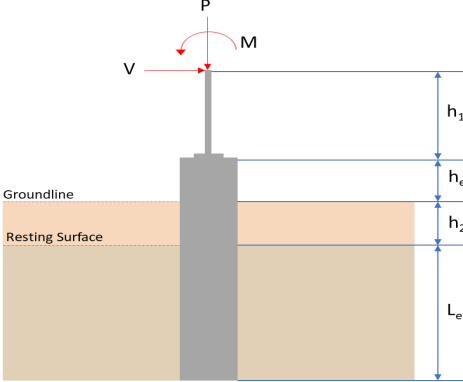
## Design Ratio

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	δ	Status
1	0.055	0.543	0.018	0.034	0.002	0.545	#16	0.420	Not Required	Pass
2	0.004	0.455	0.185	0.097	0.031	0.641	#13	0.034	Not Required	Pass
3	0.010	0.707	0.079	0.072	0.002	0.761	#13	0.044	Not Required	Pass
4	0.009	0.704	0.295	0.071	0.035	0.829	#21	0.078	Not Required	Pass
5	0.010	0.438	0.322	0.071	0.048	0.498	#21	0.073	Not Required	Pass
6	0.010	0.662	0.074	0.066	0.001	0.698	#13	0.044	Not Required	Pass
7	0.010	0.411	0.307	0.066	0.046	0.471	#21	0.073	Not Required	Pass
8	0.001	0.084	0.130	0.047	0.011	0.209	#21	0.088	Not Required	Pass
9	0.018	0.081	0.054	0.001	0.001	0.140	#13	0.198	Not Required	Pass
10	0.009	0.658	0.308	0.067	0.038	0.832	#21	0.078	Not Required	Pass
11	0.000	0.084	0.132	0.047	0.011	0.210	#21	0.088	Not Required	Pass
12	0.005	0.410	0.171	0.091	0.029	0.581	#13	0.034	Not Required	Pass
13	0.007	0.298	0.314	0.058	0.014	0.593	#21	0.265	Not Required	Pass
14	0.006	0.306	0.314	0.058	0.014	0.595	#21	0.177	Not Required	Pass
15	0.000	0.145	0.188	0.038	0.009	0.325	#21	Not Required	Not Required	Pass
16	0.000	0.145	0.188	0.038	0.009	0.325	#21	Not Required	Not Required	Pass
101	0.055	0.543	0.018	0.034	0.002	0.545	#16	0.420	Not Required	Pass
102	0.005	0.410	0.171	0.091	0.029	0.581	#13	0.034	Not Required	Pass
103	0.010	0.662	0.074	0.066	0.001	0.698	#13	0.044	Not Required	Pass
104	0.009	0.658	0.308	0.067	0.038	0.832	#21	0.078	Not Required	Pass
105	0.010	0.411	0.307	0.066	0.046	0.471	#21	0.073	Not Required	Pass
106	0.010	0.707	0.079	0.072	0.002	0.761	#13	0.044	Not Required	Pass
107	0.010	0.438	0.322	0.071	0.048	0.498	#21	0.073	Not Required	Pass
108	0.000	0.145	0.188	0.038	0.009	0.325	#21	Not Required	Not Required	Pass
109	0.018	0.081	0.054	0.001	0.001	0.140	#13	0.198	Not Required	Pass
110	0.009	0.704	0.295	0.071	0.035	0.829	#21	0.078	Not Required	Pass
111	0.000	0.145	0.188	0.038	0.009	0.325	#21	Not Required	Not Required	Pass
112	0.004	0.455	0.185	0.097	0.031	0.641	#13	0.034	Not Required	Pass
113	0.007	0.298	0.314	0.058	0.014	0.593	#21	0.177	Not Required	Pass
114	0.006	0.306	0.314	0.058	0.014	0.595	#21	0.265	Not Required	Pass
115	0.000	0.283	0.161	0.047	0.011	0.428	#21	0.557	Not Required	Pass
116	0.001	0.283	0.162	0.047	0.011	0.429	#21	0.557	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: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 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 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.714</td> <td>8.239</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.544</td> <td>-2.574</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.088</td> <td>-0.135</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.163</td> <td>-0.251</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>13.587</td> <td>22.970</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)	5.714	8.239	$V_x$ (kip)	-1.544	-2.574	$V_z$ (kip)	-0.088	-0.135	$M_x$ (kipft)	-0.163	-0.251	$M_z$ (kipft)	13.587	22.970	
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$M_x$ (kipft)	-0.163	-0.251																										
$M_z$ (kipft)	13.587	22.970																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.544 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24586 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(13.587 \text{ kipft}) + ((-1.544 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1635 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.6991 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.088 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.014013 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.163 \text{ kipft}) + ((-0.088 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025955 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0588 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.6991 \text{ ft}), (1.0588 \text{ ft})]$$

$$L_{e,req} = 4.699 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.699 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9398$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.714 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.357125 \text{ kip/ft}^2$$

$$q = 0.35713 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35713 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17856$$

Status: **PASS**  
Ratio: **0.180**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 1.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.24586 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.1635 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.1635 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.1635 \text{ kipft/ft})) + (4 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4478 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.1635 \text{ kipft/ft})) + (3 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (2.1635 \text{ kipft/ft})) + (2 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.18351 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.1635 \text{ kipft/ft})) + ((-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74346 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4478 \text{ ft})}{2}$$

$$p_a = 0.25859 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18351 \text{ kip/ft}^2)}{(0.25859 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.70966$$

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

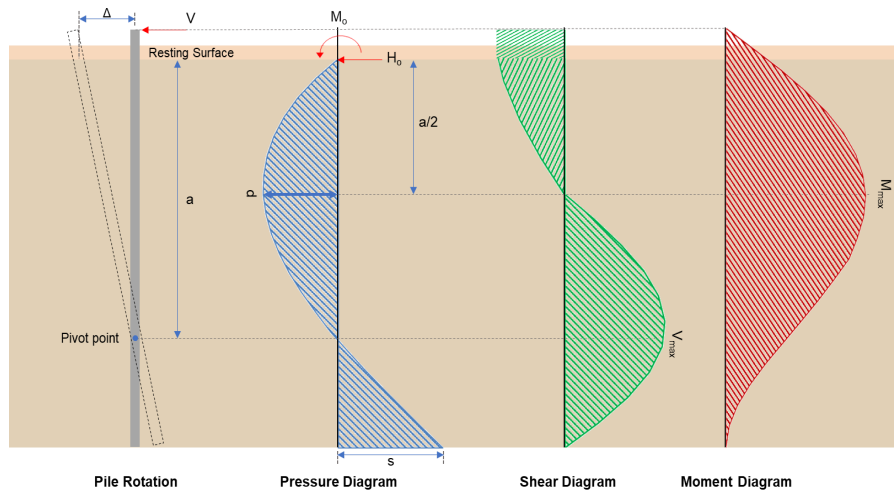
Status: **PASS**  
Ratio: **0.710**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74346 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9913$	Status: <b>PASS</b> Ratio: <b>0.990</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = -0.014013 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.025955 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.025955 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.014013 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.025955 \text{ kipft/ft})) + (4 \times (-0.014013 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.6012 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.025955 \text{ kipft/ft})) + (3 \times (-0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 [(3 \times (0.025955 \text{ kipft/ft})) + (2 \times (-0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = -0.0054518 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.025955 \text{ kipft/ft})) + ((-0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = -0.0043567 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.6012 \text{ ft})}{2}$ $p_a = 0.27009 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0054518 \text{ kip/ft}^2)}{(0.27009 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.020185$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>-0.020</b>

$$\text{Ratio} = \frac{(-0.0043567 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0058089$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.574 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.40987 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(22.97 \text{ kipft}) + ((-2.574 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.6576 \text{ kipft/ft}$$

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

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

$$E = \frac{(3.6576 \text{ kipft/ft})}{(-0.40987 \text{ kip/ft})}$$

$$E = 8.9239 \text{ ft}$$

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

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

$$a = \frac{(4 \times (3.6576 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.40987 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.6576 \text{ kipft/ft})) + (4 \times (-0.40987 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4466 \text{ ft}$$

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

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

$$V_{max} = ((-0.40987 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4466 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4466 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.2593 \text{ kip}$$

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

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

$$M_{max} = ((-0.40987 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(8.9239 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.904 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.135 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.021497 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.251 \text{ kipft}) + ((-0.135 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.039968 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.039968 \text{ kipft/ft})}{(-0.021497 \text{ kip/ft})}$$

$$E = 1.8593 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.039968 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.021497 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.039968 \text{ kipft/ft})) + (4 \times (-0.021497 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.6008 \text{ ft}$$

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

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

$$V_{max} = ((-0.021497 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.8593 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.6008 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (1.8593 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.6008 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.11413 \text{ kip}$$

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

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

$$M_{max} = ((-0.021497 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(1.8593 \text{ ft})}{(5 \text{ ft})} + \frac{(3.6008 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (1.8593 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.6008 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (1.8593 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.6008 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.24713 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.322 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.322 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.239 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0030798$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.239 \text{ kip} \rightarrow 8239 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8239 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.58 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

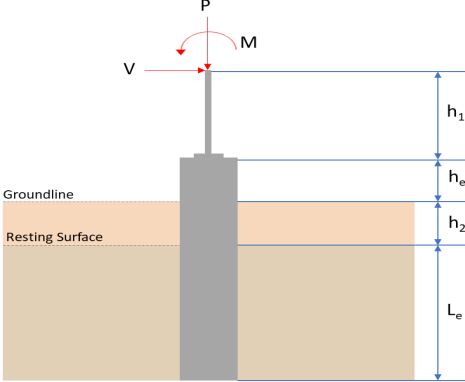
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.58 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.58 \text{ kip}$$

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.58 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.81 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 6.2593 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(6.2593 \text{ kip})}{(110.81 \text{ kip})}$ $\text{Ratio} = 0.056487$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.11413 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.11413 \text{ kip})}{(110.81 \text{ kip})}$ $\text{Ratio} = 0.00103$	<p>Status: <b>PASS</b> Ratio: <b>0.060</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),          Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 14.904 \text{ kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.904 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.05971$	<p>Status: <b>PASS</b>          Ratio: <b>0.060</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.24713 \text{ kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.24713 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0009901$	<p>Status: <b>PASS</b>          Ratio: <b>0.000</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></p> <p>Pile shape: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 5</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_n</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>5.714</td> <td>8.239</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.544</td> <td>-2.574</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.088</td> <td>0.135</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.163</td> <td>0.252</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>13.587</td> <td>22.970</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><math>f'_{ck} = 2.5</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_n$ ) (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)	5.714	8.239	$V_x$ (kip)	-1.544	-2.574	$V_z$ (kip)	0.088	0.135	$M_x$ (kipft)	0.163	0.252	$M_z$ (kipft)	13.587	22.970	
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	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><math>H</math> - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p><b>Considering x-direction:</b></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.544 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24586 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(13.587 \text{ kipft}) + ((-1.544 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.1635 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.6991 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.088 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.014013 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.163 \text{ kipft}) + ((0.088 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025955 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4937 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(4.6991 \text{ ft}), (1.4937 \text{ ft})]$$

$$L_{e,req} = 4.699 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(4.699 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9398$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.714 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.357125 \text{ kip/ft}^2$$

$$q = 0.35713 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.35713 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.17856$$

Status: **PASS**  
Ratio: **0.180**

Czerniak

**Lateral Soil Pressure (ASD):**

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

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

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

$$L/D = 1.25$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.24586 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.1635 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.1635 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.1635 \text{ kipft/ft})) + (4 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4478 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.1635 \text{ kipft/ft})) + (3 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (2.1635 \text{ kipft/ft})) + (2 \times (-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.18351 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.1635 \text{ kipft/ft})) + ((-0.24586 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74346 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4478 \text{ ft})}{2}$$

$$p_a = 0.25859 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18351 \text{ kip/ft}^2)}{(0.25859 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.70966$$

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

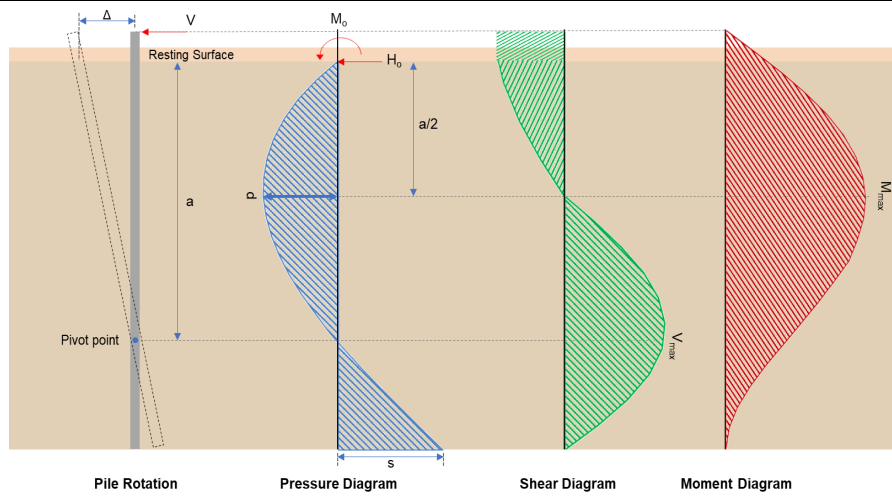
Status: **PASS**  
Ratio: **0.710**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74346 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9913$	Status: <b>PASS</b> Ratio: <b>0.990</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.014013 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.025955 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.025955 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.014013 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.025955 \text{ kipft/ft})) + (4 \times (0.014013 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.6012 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.025955 \text{ kipft/ft})) + (3 \times (0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.025955 \text{ kipft/ft})) + (2 \times (0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.01357 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.025955 \text{ kipft/ft})) + ((0.014013 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.029274 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.6012 \text{ ft})}{2}$ $p_a = 0.27009 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.01357 \text{ kip/ft}^2)}{(0.27009 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.050242$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.050</b>

$$\text{Ratio} = \frac{(0.029274 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.039032$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.574 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.40987 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(22.97 \text{ kipft}) + ((-2.574 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.6576 \text{ kipft/ft}$$

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

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

$$E = \frac{(3.6576 \text{ kipft/ft})}{(-0.40987 \text{ kip/ft})}$$

$$E = 8.9239 \text{ ft}$$

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

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

$$a = \frac{(4 \times (3.6576 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.40987 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.6576 \text{ kipft/ft})) + (4 \times (-0.40987 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4466 \text{ ft}$$

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

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

$$V_{max} = ((-0.40987 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4466 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4466 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.2593 \text{ kip}$$

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

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

$$M_{max} = ((-0.40987 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(8.9239 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (8.9239 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.4466 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 14.904 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.135 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.021497 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.252 \text{ kipft}) + ((0.135 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.040127 \text{ kipft/ft}$$

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

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

$$E = \frac{(0.040127 \text{ kipft/ft})}{(0.021497 \text{ kip/ft})}$$

$$E = 1.8667 \text{ ft}$$

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

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

$$a = \frac{(4 \times (0.040127 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.021497 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.040127 \text{ kipft/ft})) + (4 \times (0.021497 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.6004 \text{ ft}$$

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

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

$$V_{max} = ((0.021497 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (1.8667 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.6004 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (1.8667 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.6004 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.11435 \text{ kip}$$

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

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

$$M_{max} = ((0.021497 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[ \left( \frac{(1.8667 \text{ ft})}{(5 \text{ ft})} + \frac{(3.6004 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (1.8667 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left( \frac{(3.6004 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (1.8667 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left( \frac{(3.6004 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.24768 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

$f'_{ck} = 2.5 \text{ ksi}$  - Concrete strength,  
 $f_{yk} = 60 \text{ ksi}$  - Longitudinal reinforcement strength,  
 $\phi = 0.65$  - Reduction factor for axial strength,  
 $\alpha = 0.8$  - Alpha factor for axial strength,  
 $A_g = 2304 \text{ in}^2$  - Gross area of concrete,

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(8.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.322 \text{ in}^2$$

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

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

$$A_{min} = \text{Max} [(-84.322 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$n_{rebar} = 14$$

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

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

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

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

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

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

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

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.239 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0030798$$

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

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

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$ ,  $P = 8.239 \text{ kip} \rightarrow 8239 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8239 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.58 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.58 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.58 \text{ kip}$$

<p>22.5.1.2</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> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.58 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.81 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 6.2593 \text{ kip}</math> - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(6.2593 \text{ kip})}{(110.81 \text{ kip})}$ $\text{Ratio} = 0.056487$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.11435 \text{ kip}</math> - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.11435 \text{ kip})}{(110.81 \text{ kip})}$ $\text{Ratio} = 0.001032$	<p>Status: <b>PASS</b> Ratio: <b>0.060</b></p> <p>Status: <b>PASS</b> Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 14.904 \text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.904 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.05971$	<p>Status: <b>PASS</b>  Ratio: <b>0.060</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.24768 \text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.24768 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0009231$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>