

Your Project Calculations



Project Name: Dry Creek Preserve RevA

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Dry%20Creek%20Preserve%20RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=FiurUPmoECTnyPmf8fnK2zfFEVfywyjHAKdIT18VJBPYSz2BQg55nVXclqZpNI

Array Specification

Product:	Beam
Unique ID:	2P-17-8TOP-SD-24-L-4Hx4W-J007
Duty Classification:	SD
Module Width:	43.39 in
Module Length:	86.10in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	33
Front Edge Clearance:	9
Total Array Height at Tilt:	16.92 ft
Total Frame Length:	28.50 ft
Frame Weight:	1558 lbs
Array Dimensions N/S:	14.63 ft
Array Dimensions E/W:	29.03 ft
Rail Length:	175.56 in
Rail Spacing:	3.59 ft
Rail Check:	Not Checked

Support Specifications

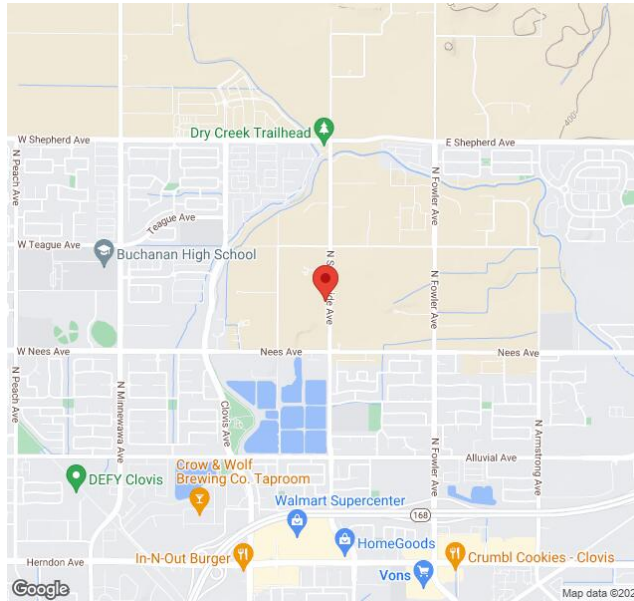
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	12.98 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.25 ft
Foundation Volume:	7.407 y ³
Foundation Result:	PASSED
Mount Twist:	0.620022 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	8195 N Sunnyside Ave, Clovis, CA 93619, USA
Wind Speed:	89 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

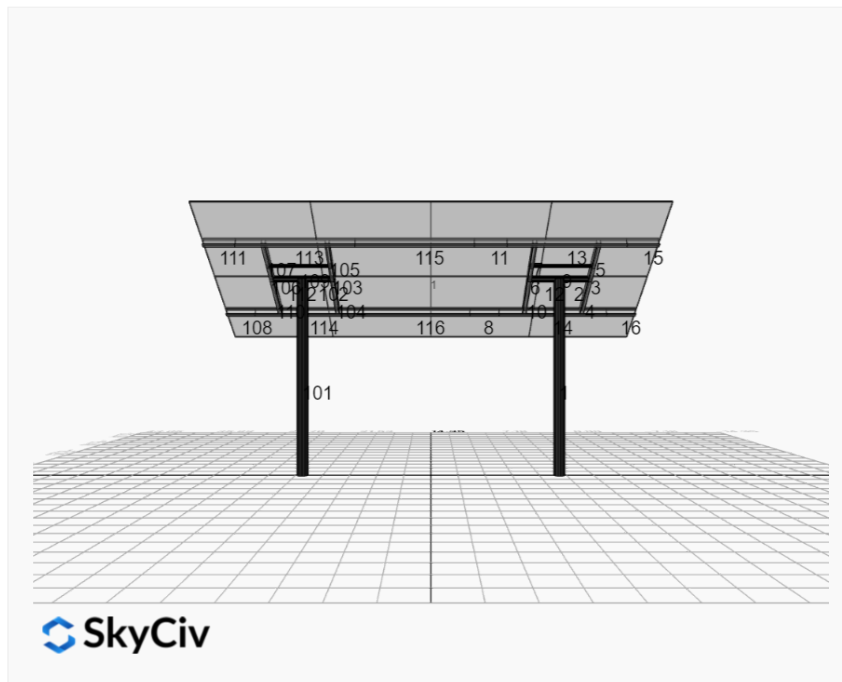
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

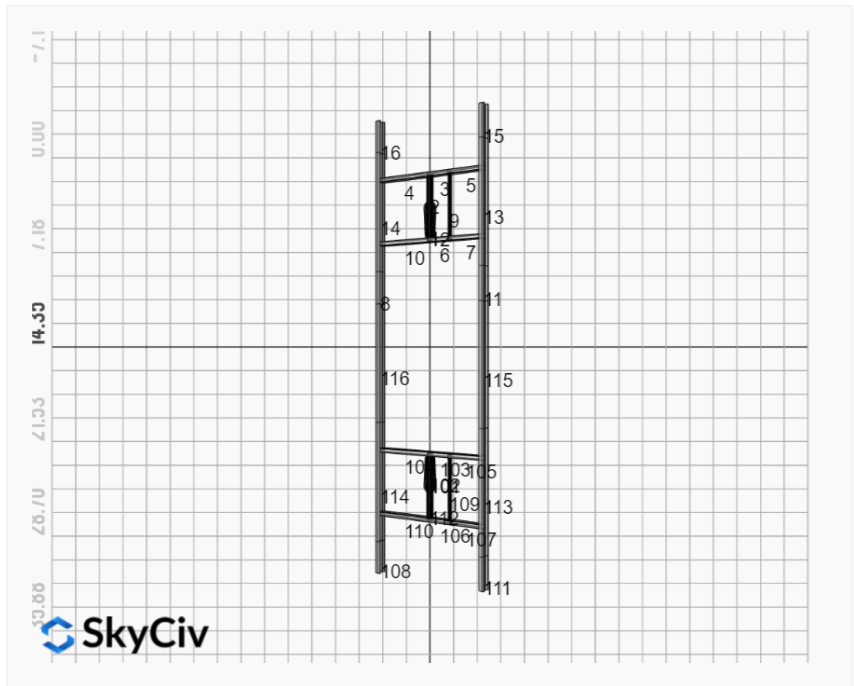
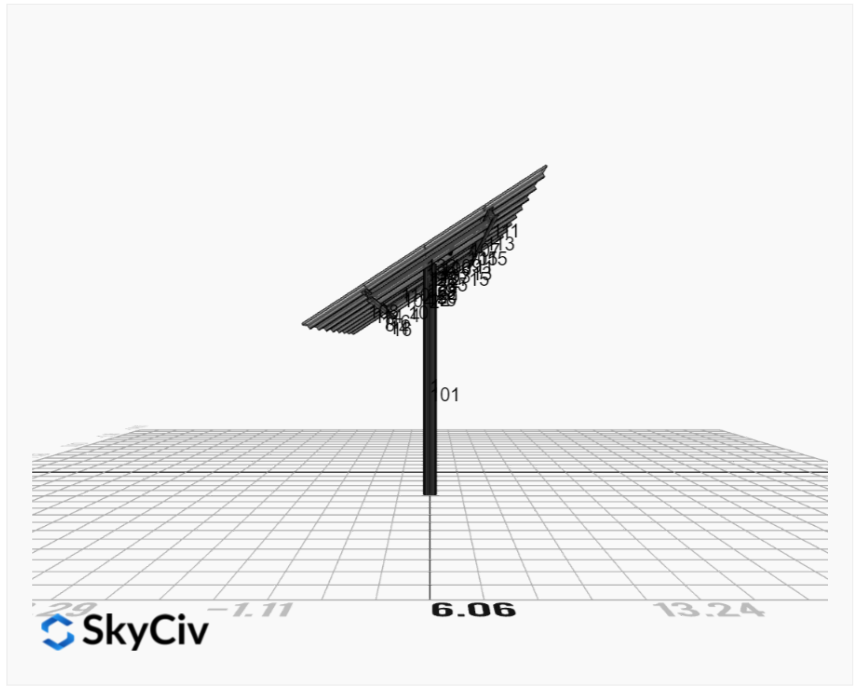
AutoDesigner Input

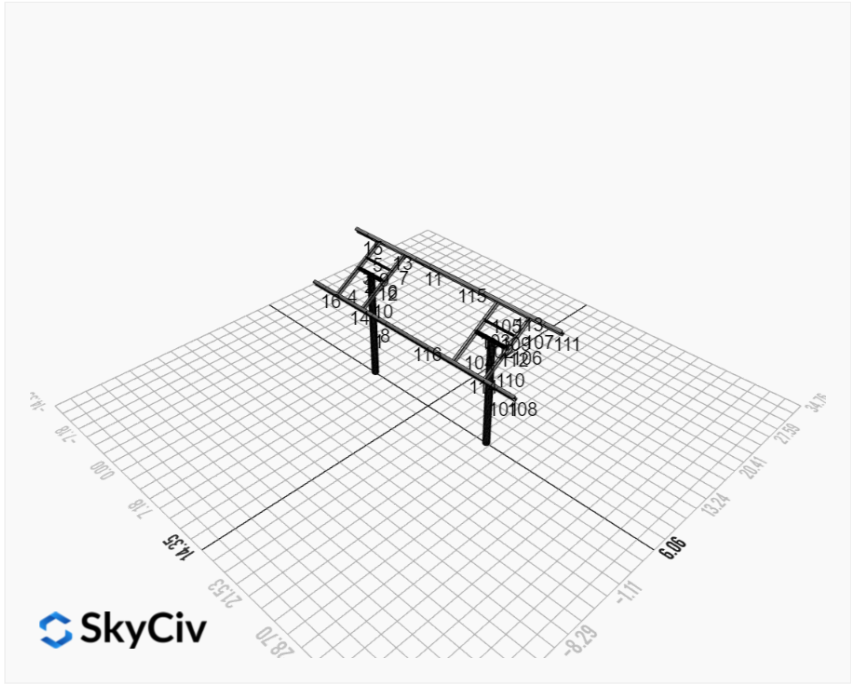
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Design Notes:

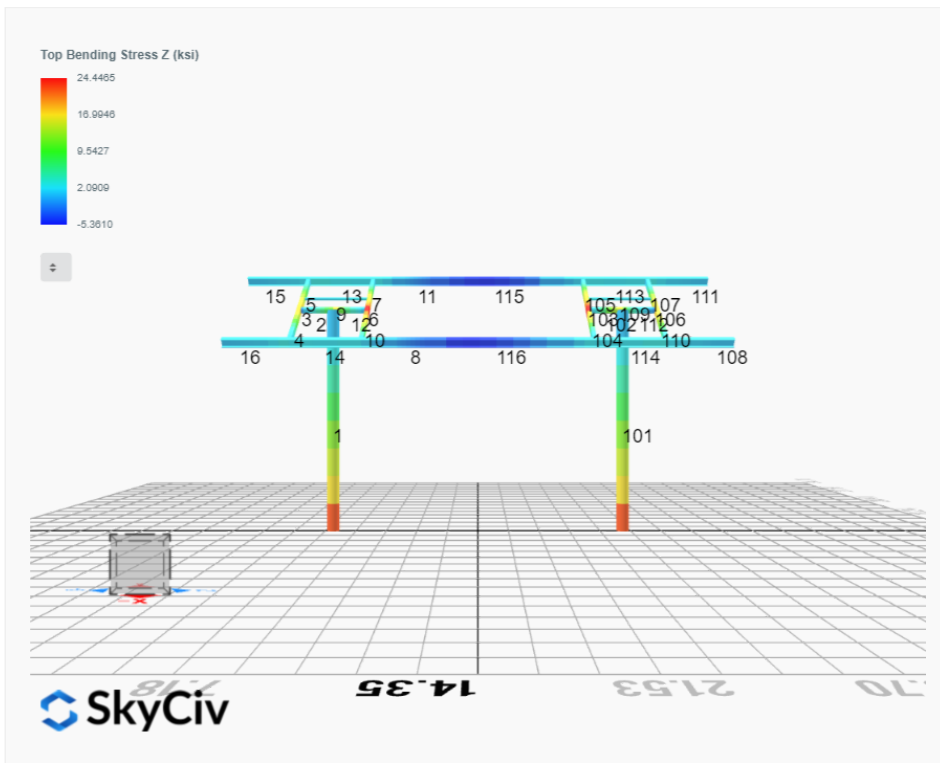
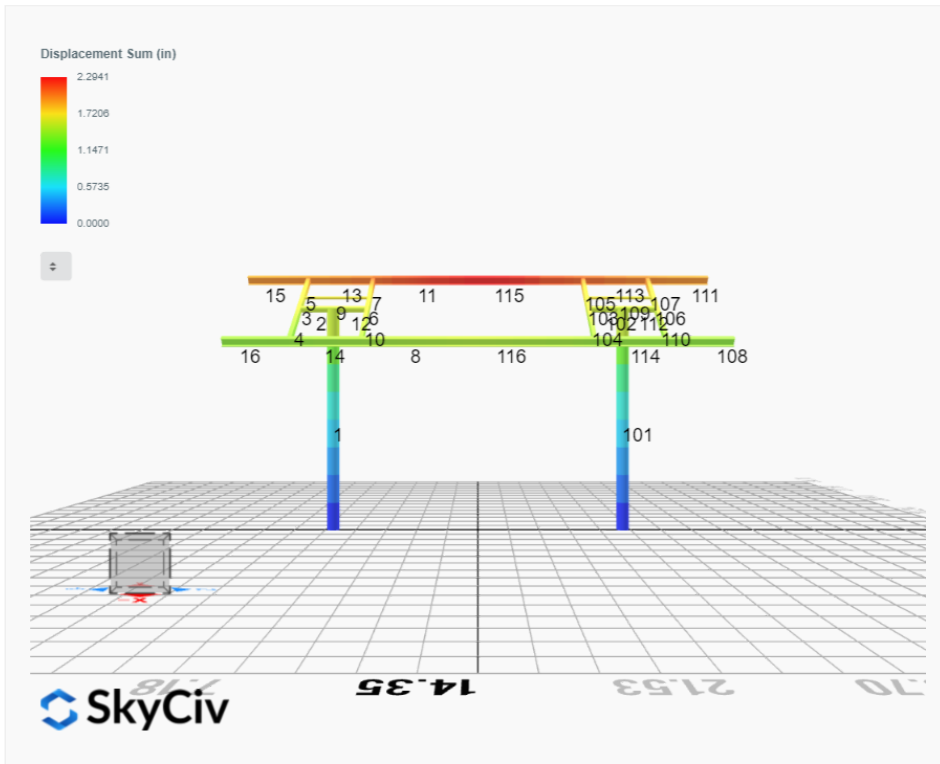
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

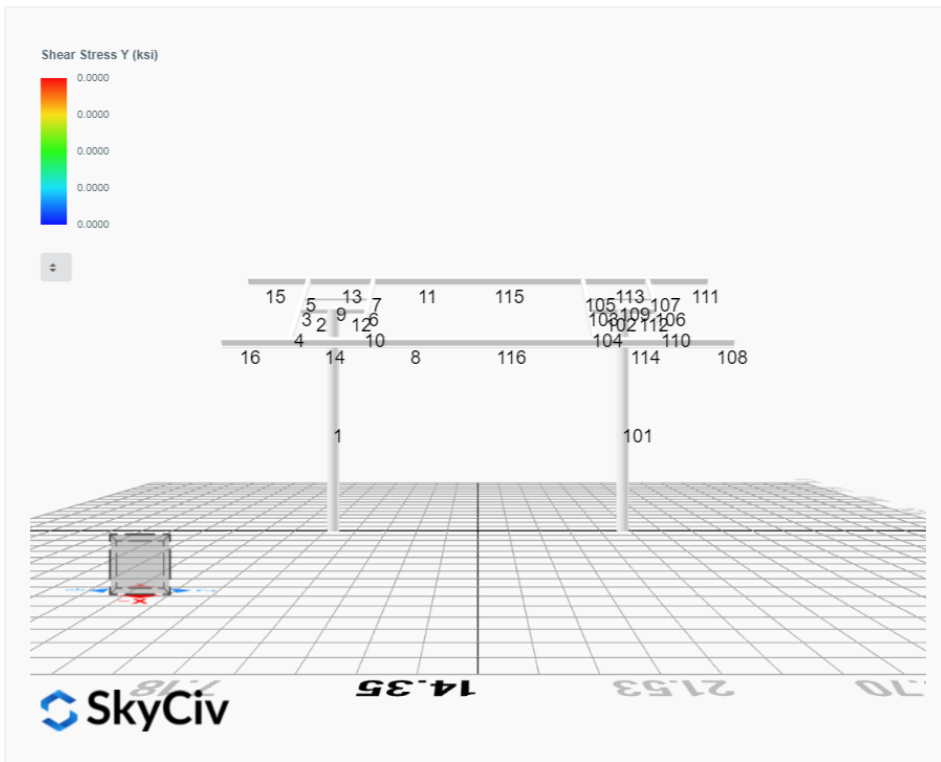
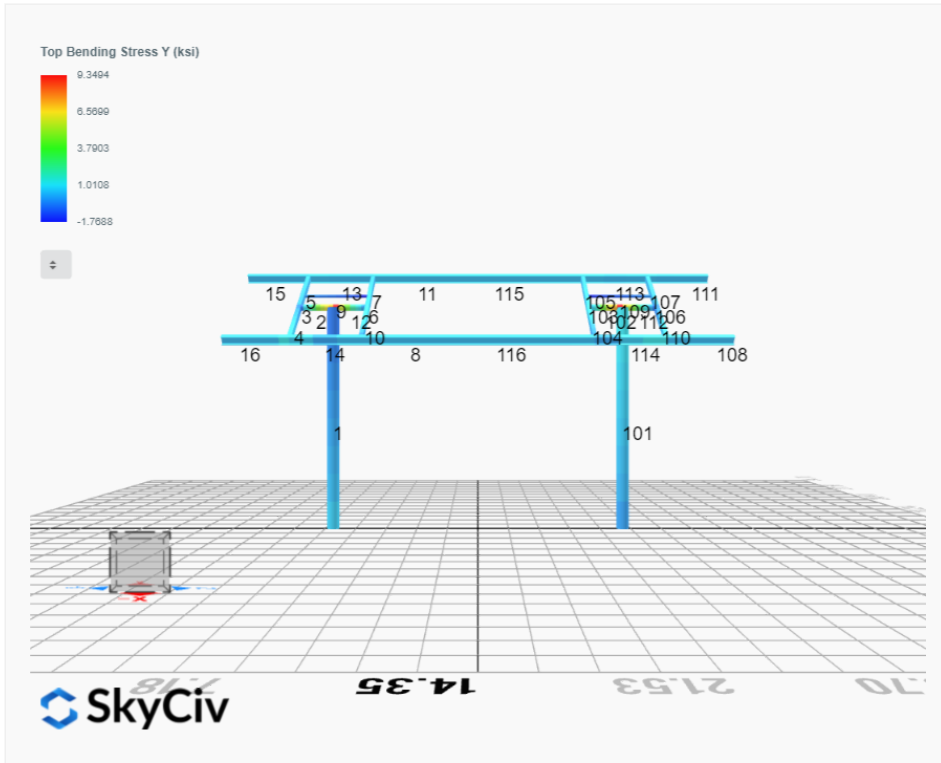


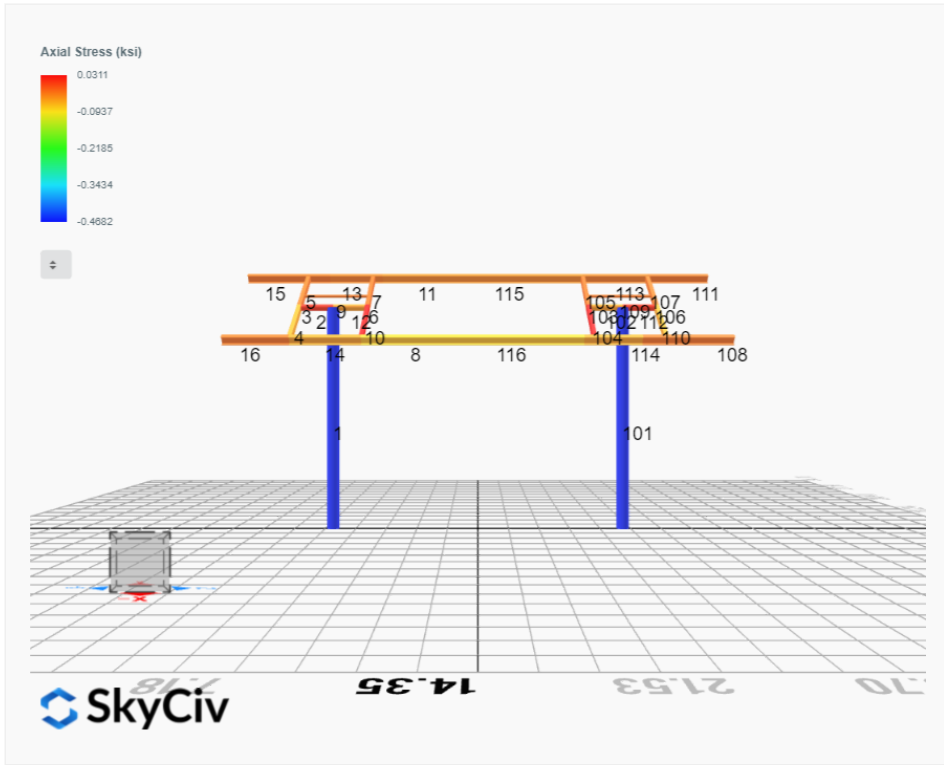




FEM Results (Envelope Worst Case for each member)







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	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 2. D + L	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 3. D + (S or Lr or R)	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 3. D + (S or Lr or R)	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 5b. D + 0.7E	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.7487	0.0421	0.1650	-0.0414	0.0192
ULS: 8. 0.6D + 0.7E	0.0000	1.0492	0.0252	0.0990	-0.0249	0.0115
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8047	4.5277	0.1551	0.5943	-0.3838	23.9991
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8047	4.5277	0.1551	0.5943	-0.3838	23.9991
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5323	-0.6108	-0.0535	-0.1968	0.2487	-19.5803
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2769	-0.2176	-0.0383	-0.1391	0.2023	-21.8763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3535	3.8329	0.1269	0.4870	-0.2982	18.0041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3535	3.8329	0.1269	0.4870	-0.2982	18.0041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1492	-0.0210	-0.0296	-0.1064	0.1762	-14.6804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9577	0.2740	-0.0182	-0.0631	0.1414	-16.4025
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3535	3.8329	0.1269	0.4870	-0.2982	18.0041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3535	3.8329	0.1269	0.4870	-0.2982	18.0041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1492	-0.0210	-0.0296	-0.1064	0.1762	-14.6804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9577	0.2740	-0.0182	-0.0631	0.1414	-16.4025
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8047	3.8282	0.1383	0.5283	-0.3672	23.9914
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8047	3.8282	0.1383	0.5283	-0.3672	23.9914
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5323	-1.3103	-0.0704	-0.2628	0.2653	-19.5880
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2769	-0.9171	-0.0551	-0.2051	0.2189	-21.8840

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	6.7301
Shear X	-3.0078
Shear Z	0.2391
Moment X	0.9147
Moment Y (Twist)	0.6201
Moment Z	40.3455

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.5277
Shear X	-1.8047
Shear Z	0.1551
Moment X	0.5943
Moment Y (Twist)	0.3838
Moment Z	23.9991

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	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 2. D + L	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 3. D + (S or Lr or R)	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 3. D + (S or Lr or R)	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 5b. D + 0.7E	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	1.7487	-0.0421	-0.1650	0.0415	0.0192
ULS: 8. 0.6D + 0.7E	-0.0000	1.0492	-0.0252	-0.0990	0.0249	0.0115
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8047	4.5277	-0.1551	-0.5943	0.3838	23.9991
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8047	4.5277	-0.1551	-0.5943	0.3838	23.9991
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5323	-0.6108	0.0535	0.1968	-0.2487	-19.5803
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2769	-0.2176	0.0383	0.1391	-0.2023	-21.8763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3535	3.8329	-0.1269	-0.4870	0.2982	18.0042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3535	3.8329	-0.1269	-0.4870	0.2982	18.0042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1492	-0.0210	0.0296	0.1064	-0.1762	-14.6804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9577	0.2740	0.0182	0.0631	-0.1414	-16.4024
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3535	3.8329	-0.1269	-0.4870	0.2982	18.0042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3535	3.8329	-0.1269	-0.4870	0.2982	18.0042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1492	-0.0210	0.0296	0.1064	-0.1762	-14.6804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9577	0.2740	0.0182	0.0631	-0.1414	-16.4024
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8047	3.8282	-0.1383	-0.5283	0.3672	23.9914
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8047	3.8282	-0.1383	-0.5283	0.3672	23.9914
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5323	-1.3103	0.0704	0.2628	-0.2653	-19.5880
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2769	-0.9171	0.0551	0.2051	-0.2189	-21.8840

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	6.7301
Shear X	-3.0078
Shear Z	-0.2391
Moment X	-0.9146
Moment Y (Twist)	0.6200
Moment Z	40.3459

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.5277
Shear X	-1.8047
Shear Z	-0.1551
Moment X	-0.5943
Moment Y (Twist)	0.3838
Moment Z	23.9991

Project Details

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

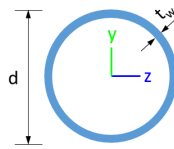


Design Input Information

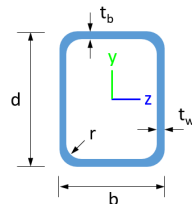
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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				
9	8in Pipe Sch 40	8.63	0.32				



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 ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

115	120.00	89.27	18.73	6.45	30.09	45.74
116	120.60	89.27	18.73	6.45	30.09	45.74

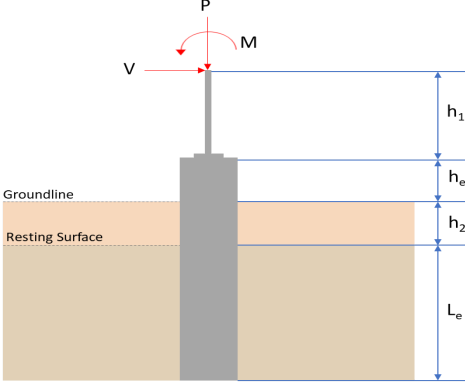
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.044	0.484	0.026	0.027	0.002	0.517	#13	0.557	Not Required	Pass
2	0.001	0.268	0.157	0.063	0.033	0.425	#13	0.052	Not Required	Pass
3	0.003	0.519	0.031	0.051	0.004	0.532	#13	0.044	Not Required	Pass
4	0.003	0.509	0.065	0.051	0.009	0.575	#13	0.078	Not Required	Pass
5	0.003	0.322	0.029	0.052	0.004	0.327	#13	0.073	Not Required	Pass
6	0.004	0.657	0.061	0.067	0.010	0.719	#13	0.044	Not Required	Pass
7	0.004	0.407	0.069	0.066	0.010	0.425	#13	0.073	Not Required	Pass
8	0.001	0.115	0.022	0.037	0.003	0.135	#13	0.088	Not Required	Pass
9	0.000	0.063	0.050	0.002	0.002	0.113	#13	0.198	Not Required	Pass
10	0.004	0.645	0.076	0.065	0.011	0.653	#13	0.078	Not Required	Pass
11	0.001	0.116	0.021	0.038	0.003	0.133	#13	0.088	Not Required	Pass
12	0.001	0.403	0.196	0.084	0.038	0.599	#13	0.167	Not Required	Pass
13	0.001	0.100	0.068	0.052	0.005	0.130	#13	0.265	Not Required	Pass
14	0.002	0.102	0.067	0.051	0.005	0.125	#13	0.177	Not Required	Pass
15	0.000	0.020	0.010	0.016	0.001	0.029	#13	Not Required	Not Required	Pass
16	0.000	0.020	0.010	0.016	0.001	0.029	#13	Not Required	Not Required	Pass
101	0.044	0.484	0.026	0.027	0.002	0.517	#13	0.557	Not Required	Pass
102	0.001	0.403	0.196	0.084	0.038	0.599	#13	0.167	Not Required	Pass
103	0.004	0.657	0.061	0.067	0.010	0.719	#13	0.044	Not Required	Pass
104	0.004	0.645	0.076	0.065	0.011	0.653	#13	0.078	Not Required	Pass
105	0.004	0.407	0.069	0.066	0.010	0.425	#13	0.073	Not Required	Pass
106	0.003	0.519	0.031	0.051	0.004	0.532	#13	0.044	Not Required	Pass
107	0.003	0.322	0.029	0.052	0.004	0.327	#13	0.073	Not Required	Pass
108	0.000	0.020	0.010	0.016	0.001	0.029	#13	Not Required	Not Required	Pass
109	0.000	0.063	0.050	0.002	0.002	0.113	#13	0.198	Not Required	Pass
110	0.003	0.509	0.065	0.051	0.009	0.575	#13	0.078	Not Required	Pass
111	0.000	0.020	0.010	0.016	0.001	0.029	#13	Not Required	Not Required	Pass
112	0.001	0.268	0.157	0.063	0.033	0.425	#13	0.052	Not Required	Pass
113	0.001	0.100	0.068	0.052	0.005	0.130	#13	0.177	Not Required	Pass
114	0.002	0.102	0.067	0.051	0.005	0.125	#13	0.265	Not Required	Pass
115	0.001	0.186	0.040	0.038	0.003	0.219	#13	0.321	Not Required	Pass
116	0.001	0.184	0.040	0.037	0.003	0.220	#13	0.321	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F _y	Specified minimum yield stress
F _u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I _{yp}	Moment of inertia about the Y axes
I _{zp}	Moment of inertia about the Z axes
I _w	Warping constant
S _{yp}	Plastic section modulus about the Y axis
S _{zp}	Plastic section modulus about the Z axis
KL	Effective length
C _b	Buckling modification factor (from all load combinations)
L _b	Length between braced points

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
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.528</td> <td>6.730</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.805</td> <td>-3.008</td> </tr> <tr> <td>V_z (kip)</td> <td>0.155</td> <td>0.239</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.594</td> <td>0.915</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.999</td> <td>40.345</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.528	6.730	V_x (kip)	-1.805	-3.008	V_z (kip)	0.155	0.239	M_x (kipft)	0.594	0.915	M_z (kipft)	23.999	40.345	
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	<p>Required depth to resist lateral loads (ASD) H - 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>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.805 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.28742 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24 \text{ kipft}) + ((-1.805 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8215 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.8884 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.024682 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.594 \text{ kipft}) + ((0.155 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.094586 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2135 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8884 \text{ ft}), (2.2135 \text{ ft})]$$

$$L_{e,req} = 5.888 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.888 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94208$$

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

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

$$q = \frac{(4.528 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2829 \text{ kip/ft}^2$$

$$q = 0.200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.283 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.1415$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.28742 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.8215 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8215 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8215 \text{ kipft/ft})) + (4 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2909 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8215 \text{ kipft/ft})) + (3 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.8215 \text{ kipft/ft})) + (2 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.2389 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.8215 \text{ kipft/ft})) + ((-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89804 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2909 \text{ ft})}{2}$$

$$p_a = 0.32182 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2389 \text{ kip/ft}^2)}{(0.32182 \text{ kip/ft}^2)}$$

$$Ratio = 0.74235$$

p_s - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.740**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89804 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95791$$

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = 0.024682 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.094586 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.094586 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.024682 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.094586 \text{ kipft/ft})) + (4 \times (0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.438 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.094586 \text{ kipft/ft})) + (3 \times (0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.094586 \text{ kipft/ft})) + (2 \times (0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.022935 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.094586 \text{ kipft/ft})) + ((0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.052751 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.438 \text{ ft})}{2}$$

$$p_a = 0.33285 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.022935 \text{ kip/ft}^2)}{(0.33285 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.068905$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

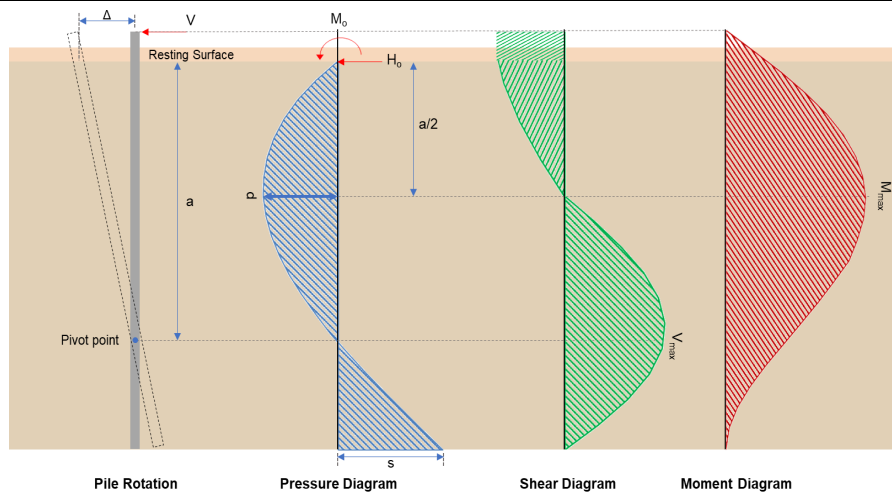
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.052751 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.056268$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.345 \text{ kipft}) + ((-3.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.4244 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.4244 \text{ kipft/ft})}{(-0.47898 \text{ kip/ft})}$$

$$E = 13.413 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.4244 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.47898 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.4244 \text{ kipft/ft})) + (4 \times (-0.47898 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2901 \text{ ft}$$

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

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

$$V_{max} = ((-0.47898 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2901 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2901 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.5413 \text{ kip}$$

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

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

$$M_{max} = ((-0.47898 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(13.413 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 25.601 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.239 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.038057 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.915 \text{ kipft}) + ((0.239 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1457 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1457 \text{ kipft/ft})}{(0.038057 \text{ kip/ft})}$$

$$E = 3.8285 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1457 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.038057 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.1457 \text{ kipft/ft})) + (4 \times (0.038057 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4381 \text{ ft}$$

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

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

$$V_{max} = ((0.038057 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4381 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4381 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.26613 \text{ kip}$$

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

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

$$M_{max} = ((0.038057 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.8285 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.74654 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.73 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.04 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.04 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: 14 - #5 (0.625 in)

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.73 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0021141$$

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

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.73 \text{ kip} \rightarrow 6730 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(6730 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.69 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

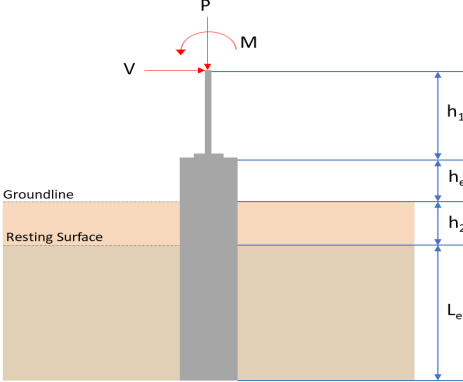
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.69 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.69 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - 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 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.69 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.03 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.5413 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.5413 \text{ kip})}{(118.03 \text{ kip})}$ $\text{Ratio} = 0.072365$ <p>Considering z-direction:</p> <p>$V_{max} = 0.26613 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.26613 \text{ kip})}{(118.03 \text{ kip})}$ $\text{Ratio} = 0.0022547$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - 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>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 25.601\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.601\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.093633$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.74654\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.74654\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0027304$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.528</td> <td>6.730</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.805</td> <td>-3.008</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.155</td> <td>-0.239</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.594</td> <td>-0.915</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.999</td> <td>40.346</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	4.528	6.730	V_x (kip)	-1.805	-3.008	V_z (kip)	-0.155	-0.239	M_x (kipft)	-0.594	-0.915	M_z (kipft)	23.999	40.346	
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	<p>Required depth to resist lateral loads (ASD) H - 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>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.805 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.28742 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(24 \text{ kipft}) + ((-1.805 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8215 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.8884 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.155 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.024682 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.594 \text{ kipft}) + ((-0.155 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.094586 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7134 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8884 \text{ ft}), (1.7134 \text{ ft})]$$

$$L_{e,req} = 5.888 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.888 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94208$$

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

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

$$q = \frac{(4.528 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.2829 \text{ kip/ft}^2$$

$$q = 0.200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.283 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.1415$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.28742 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.8215 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8215 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.8215 \text{ kipft/ft})) + (4 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2909 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8215 \text{ kipft/ft})) + (3 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.8215 \text{ kipft/ft})) + (2 \times (-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.2389 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.8215 \text{ kipft/ft})) + ((-0.28742 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89804 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2909 \text{ ft})}{2}$$

$$p_a = 0.32182 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2389 \text{ kip/ft}^2)}{(0.32182 \text{ kip/ft}^2)}$$

$$Ratio = 0.74235$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.740**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89804 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95791$$

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = -0.024682 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.094586 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.094586 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.024682 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.094586 \text{ kipft/ft})) + (4 \times (-0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.438 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.094586 \text{ kipft/ft})) + (3 \times (-0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.094586 \text{ kipft/ft})) + (2 \times (-0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0055281 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.094586 \text{ kipft/ft})) + ((-0.024682 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0053625 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.438 \text{ ft})}{2}$$

$$p_a = 0.33285 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0055281 \text{ kip/ft}^2)}{(0.33285 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.016608$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

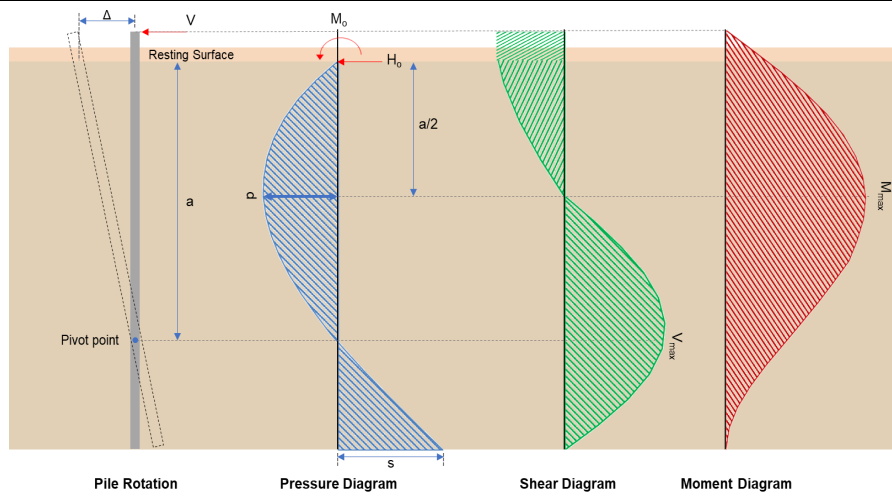
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.0053625 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0057201$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47898 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(40.346 \text{ kipft}) + ((-3.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.4245 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.4245 \text{ kipft/ft})}{(-0.47898 \text{ kip/ft})}$$

$$E = 13.413 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.4245 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.47898 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.4245 \text{ kipft/ft})) + (4 \times (-0.47898 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2901 \text{ ft}$$

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

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

$$V_{max} = ((-0.47898 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2901 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2901 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.5415 \text{ kip}$$

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

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

$$M_{max} = ((-0.47898 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(13.413 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.413 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2901 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.602 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.239 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.038057 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.915 \text{ kipft}) + ((-0.239 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1457 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1457 \text{ kipft/ft})}{(-0.038057 \text{ kip/ft})}$$

$$E = 3.8285 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1457 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.038057 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.1457 \text{ kipft/ft})) + (4 \times (-0.038057 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4381 \text{ ft}$$

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

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

$$V_{max} = ((-0.038057 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4381 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4381 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.26613 \text{ kip}$$

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

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

$$M_{max} = ((-0.038057 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.8285 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.8285 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4381 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.74654 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.73 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -102.04 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-102.04 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.73 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0021141$$

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

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.73 \text{ kip} \rightarrow 6730 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(6730 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 130.69 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (130.69 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 130.69 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - 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 $V_{s,b}$ - 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>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((130.69 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.03 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.5415 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.5415 \text{ kip})}{(118.03 \text{ kip})}$ $\text{Ratio} = 0.072367$ <p>Considering z-direction:</p> <p>$V_{max} = 0.26613 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.26613 \text{ kip})}{(118.03 \text{ kip})}$ $\text{Ratio} = 0.0022547$	<p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - 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>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 25.602\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.602\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.093635$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.74654\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.74654\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0027304$	<p>Status: PASS Ratio: 0.000</p>