

Project Details

Project Name: test14

Date: Wed Nov 06 2024

Location: 28385 Post Office Rd, Mass City, MI 49948,

Number of Modules: 30

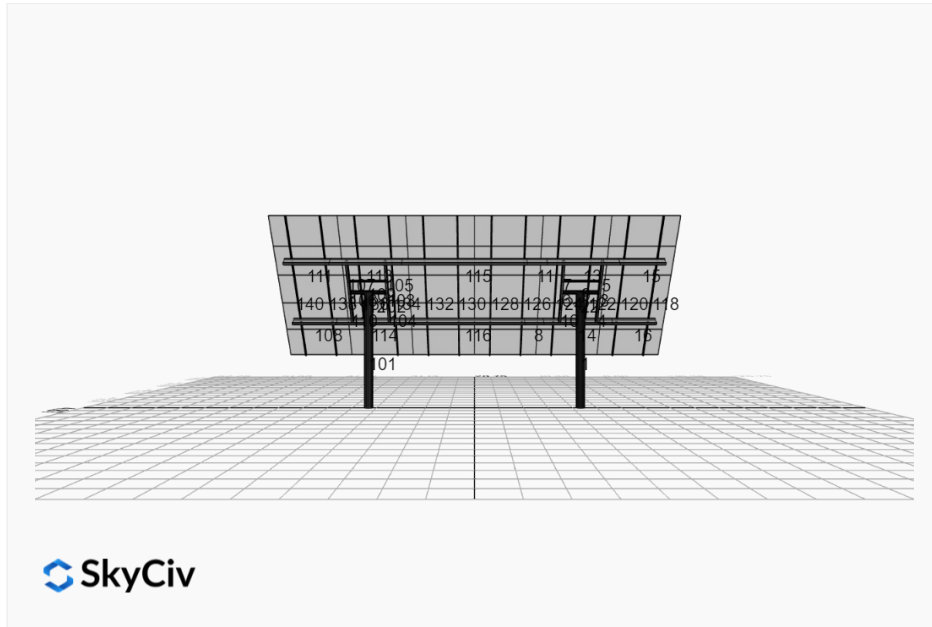
USA

Number of Poles: 2

Unique ID: 2P-22.5-10TOP-XD-57-L-5Hx6W-2CJG

Date Sold:

Dealer: _____



Array Dimensions N/S	17.42 ft
Array Dimensions E/W	41.50 ft
Winter Tilt Angle	56
Front Edge Clearance	5 ft

MT Solar Bill of Materials (2P-22.5-10TOP-XD-57-L-5Hx6W-2CJG)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	2
MTS-HF-XD	H-Frame Assembly-XD	2
MTS-XD-Wing-57	57IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	6

Rail Bill of Materials

	Part	Qty
	Rails (207in)	12
	Rail Attachment	48
	Module Mid Clamp	48
	Module End Clamp	24
	Ground Lug	6

Site Details:



Site Address: 28385 Post Office Rd, Mass City, MI 49948, USA

Array Specification

Duty Classification:	XD
Module Width:	41.30 in
Module Length:	82.00 in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Winter Tilt Angle:	56
Front Edge Clearance:	5
Total Array Height at Tilt:	19.44 ft
Total Frame Length:	39.50 ft
Frame Weight:	3072 lbs
Array Dimensions N/S:	17.42 ft
Array Dimensions E/W:	41.50 ft
Rail Length:	209.00 in
Rail Spacing:	3.46 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	12.22 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.75 ft Pile 2: 8.75 ft
Foundation Volume:	10.370 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	28385 Post Office Rd, Mass City, MI 49948, USA
Wind Speed:	120 mph
Snow Load:	80 psf

Design Disclaimer

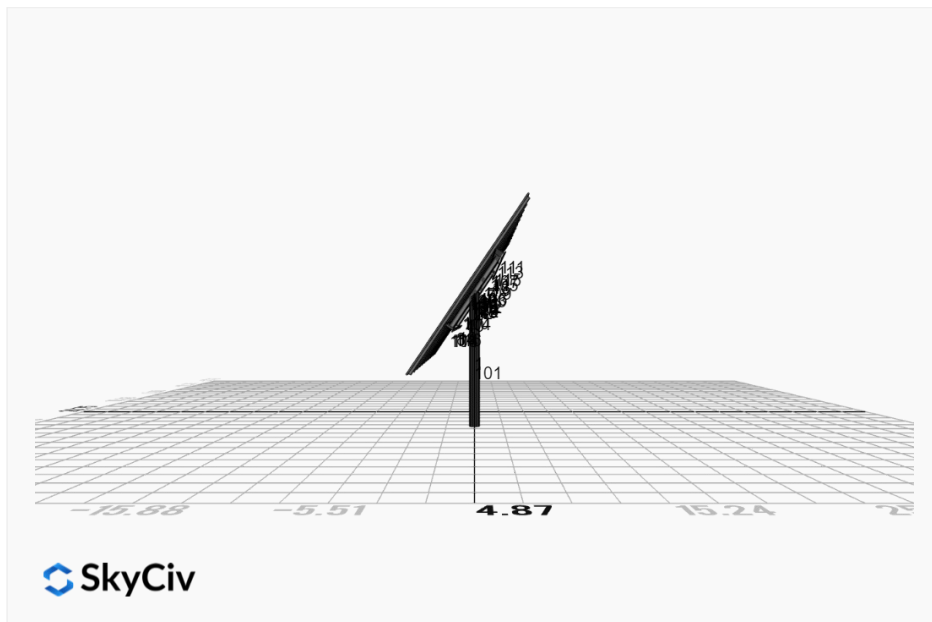
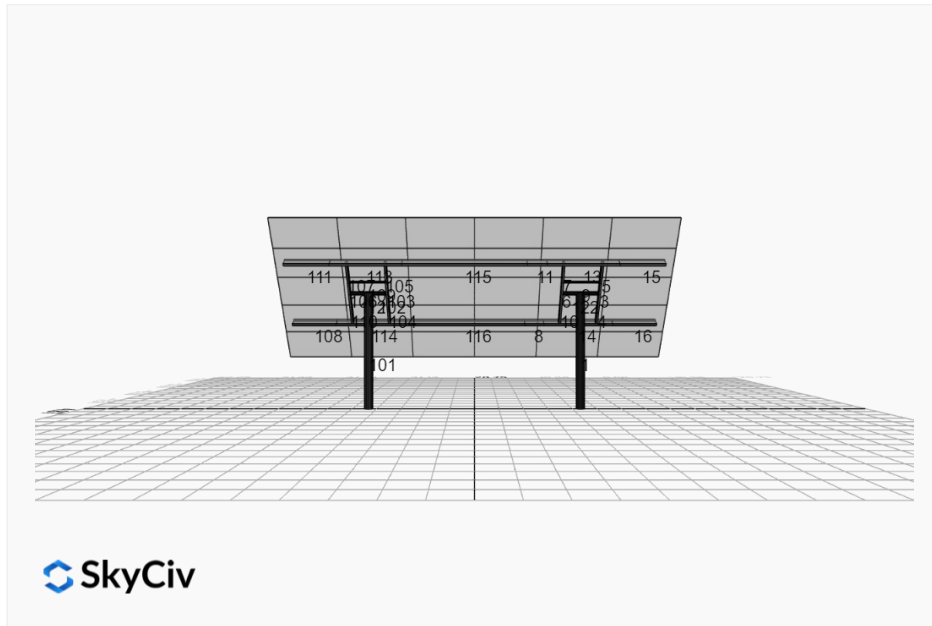
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

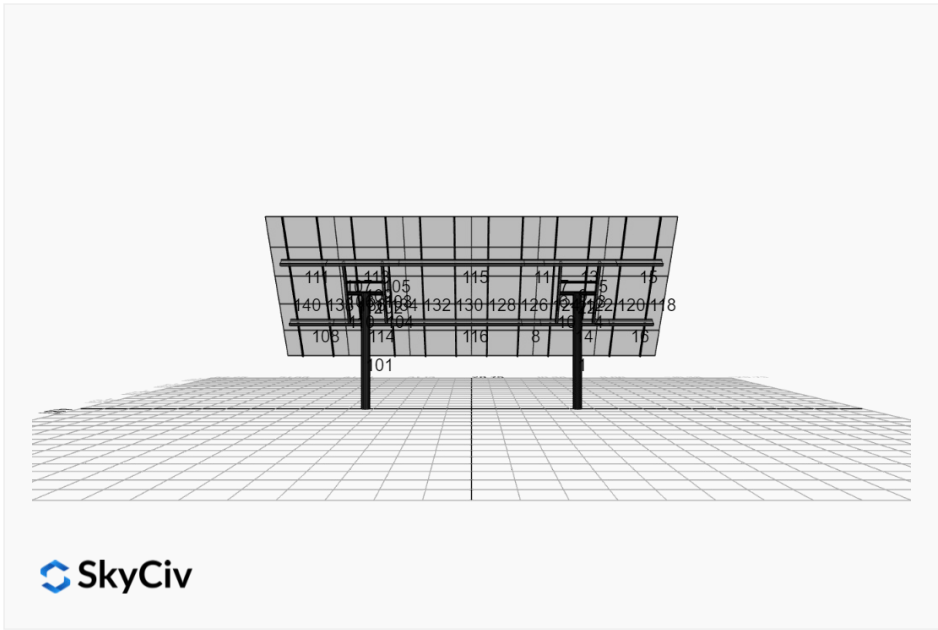
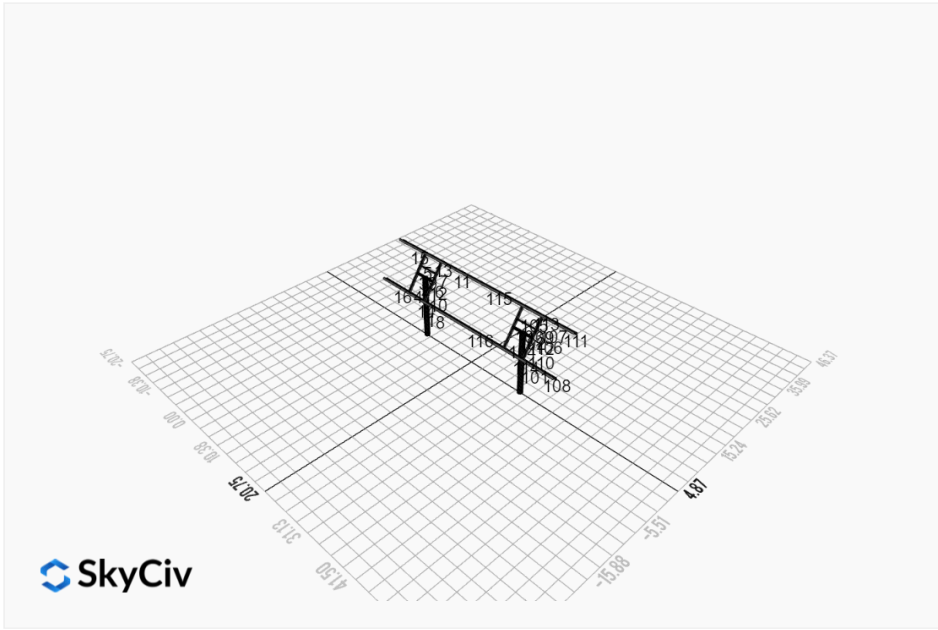
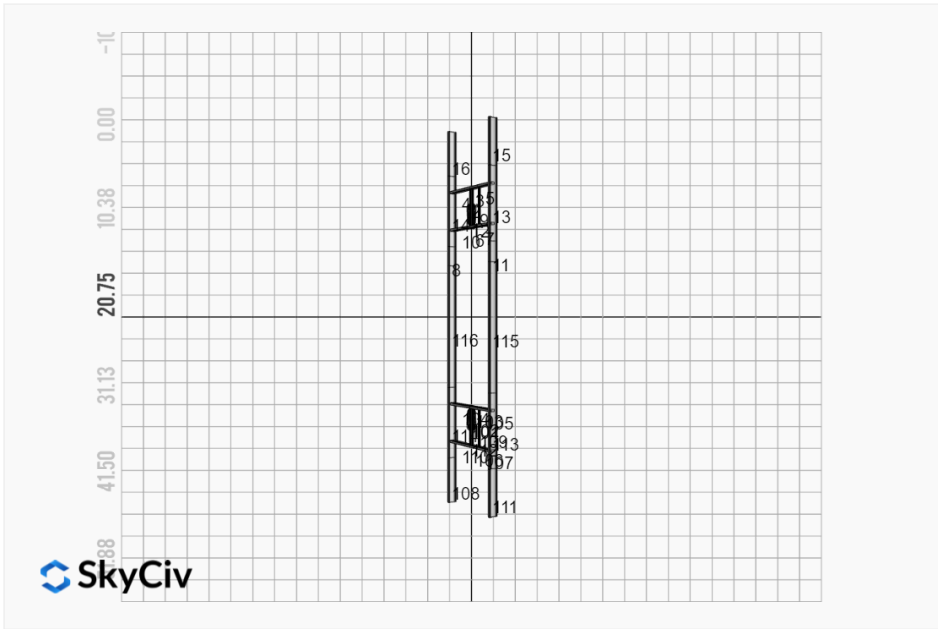
AutoDesigner Input

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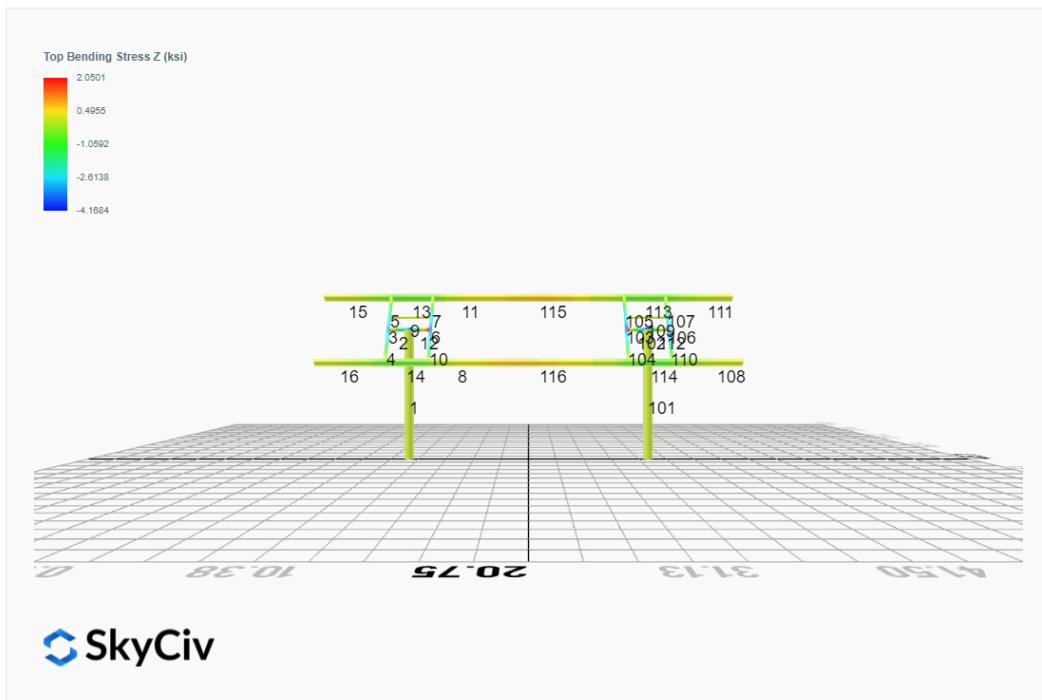
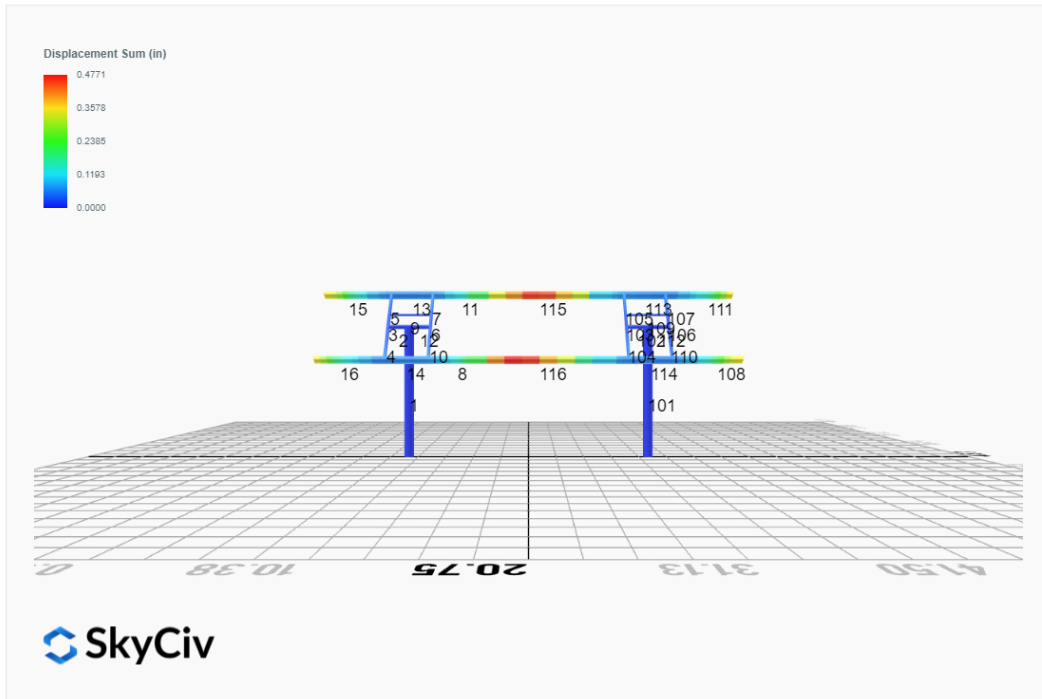
Design Notes:

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

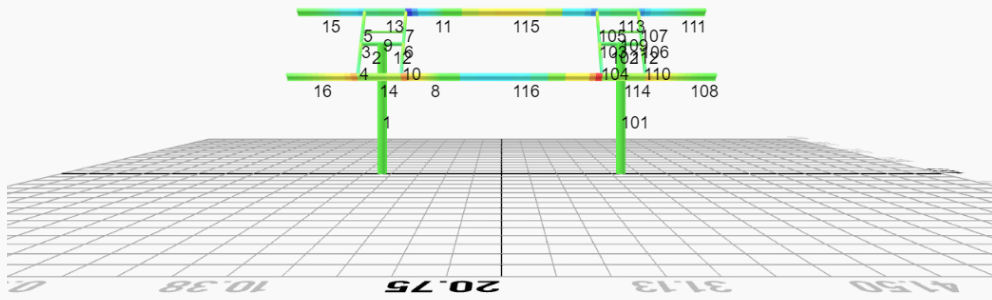
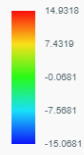




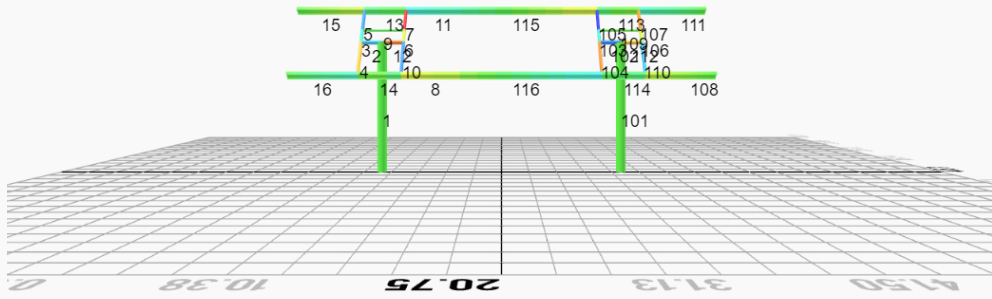
FEM Results (Envelope Worst Case for each member)

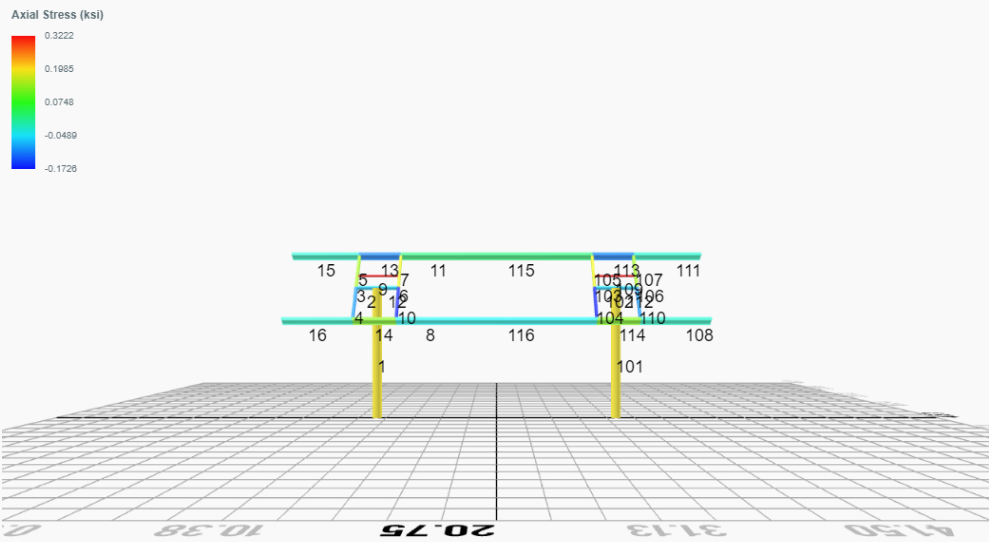


Top Bending Stress Y (ksi)



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	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 2. D + L	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 3. D + (S or Lr or R)	0.0000	5.2362	0.0824	0.2923	-0.0279	0.0262
ULS: 3. D + (S or Lr or R)	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.6439	0.0715	0.2534	-0.0241	0.0249
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 5b. D + 0.7E	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.6439	0.0715	0.2534	-0.0241	0.0249
ULS: 8. 0.6D + 0.7E	0.0000	1.7203	0.0232	0.0821	-0.0077	0.0125
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.0852	6.9717	0.1286	0.4183	-0.7438	75.1718
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.0852	-1.2373	-0.0509	-0.1432	0.7178	-73.5653
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.5639	7.7223	0.1389	0.4645	-0.5724	56.3881
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.6439	0.0715	0.2534	-0.0241	0.0249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.5639	1.5655	0.0043	0.0434	0.5238	-55.1648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.6439	0.0715	0.2534	-0.0241	0.0249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.5639	5.9456	0.1061	0.3479	-0.5610	56.3840
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.5639	-0.2112	-0.0285	-0.0732	0.5352	-55.1688
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.8672	0.0387	0.1369	-0.0128	0.0209
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.0852	5.8249	0.1131	0.3635	-0.7387	75.1634
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.7203	0.0232	0.0821	-0.0077	0.0125
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.0852	-2.3842	-0.0664	-0.1979	0.7229	-73.5737
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.7203	0.0232	0.0821	-0.0077	0.0125

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	11.4660
Shear X	-10.1420
Shear Z	0.2180
Moment X	0.7113
Moment Y (Twist)	1.2369
Moment Z	126.2817

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	7.7223
Shear X	-6.0852
Shear Z	0.1389
Moment X	0.4645
Moment Y (Twist)	0.7438
Moment Z	75.1718

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	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 2. D + L	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 3. D + (S or Lr or R)	-0.0000	5.2362	-0.0824	-0.2924	0.0280	0.0263
ULS: 3. D + (S or Lr or R)	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.6439	-0.0715	-0.2535	0.0242	0.0249
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 5b. D + 0.7E	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.6439	-0.0715	-0.2535	0.0242	0.0249
ULS: 8. 0.6D + 0.7E	-0.0000	1.7203	-0.0232	-0.0821	0.0077	0.0125
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.0852	6.9717	-0.1286	-0.4183	0.7438	75.1718
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.0852	-1.2373	0.0509	0.1431	-0.7178	-73.5653
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.5639	7.7223	-0.1389	-0.4646	0.5724	56.3881
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	4.6439	-0.0715	-0.2535	0.0242	0.0249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.5639	1.5655	-0.0043	-0.0435	-0.5238	-55.1647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	4.6439	-0.0715	-0.2535	0.0242	0.0249
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.5639	5.9456	-0.1061	-0.3479	0.5611	56.3841
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.5639	-0.2112	0.0285	0.0731	-0.5351	-55.1687
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.8672	-0.0387	-0.1369	0.0128	0.0209
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.0852	5.8249	-0.1131	-0.3635	0.7387	75.1634
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.7203	-0.0232	-0.0821	0.0077	0.0125
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.0852	-2.3842	0.0664	0.1979	-0.7229	-73.5736
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.7203	-0.0232	-0.0821	0.0077	0.0125

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	11.4660
Shear X	-10.1421
Shear Z	-0.2180
Moment X	-0.7118
Moment Y (Twist)	1.2372
Moment Z	126.2835

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	7.7223
Shear X	-6.0852
Shear Z	-0.1389
Moment X	-0.4646
Moment Y (Twist)	0.7438
Moment Z	75.1718

Project Details

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 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial

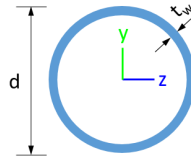


Design Input Information

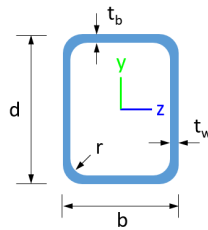
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Design Materials			
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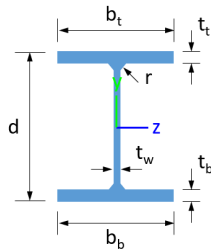
Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
11	10in Pipe Sch 40	10.75	0.36				



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



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

Section Properties

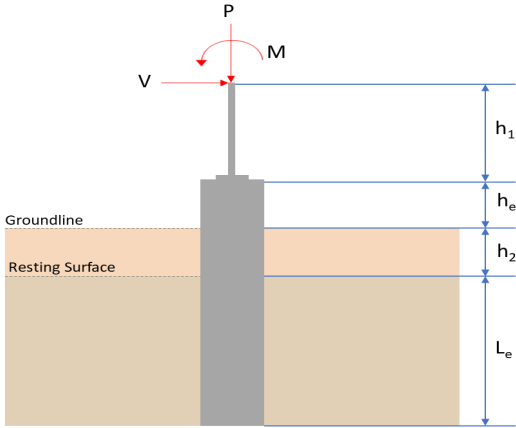
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
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5	0.007	0.395	0.161	0.063	0.043	0.420	#13	0.076	Not Required	Pass
6	0.010	0.710	0.066	0.071	0.008	0.755	#13	0.046	Not Required	Pass
7	0.010	0.440	0.214	0.070	0.056	0.474	#13	0.076	Not Required	Pass
8	0.002	0.073	0.264	0.048	0.022	0.285	#21	0.102	Not Required	Pass
9	0.021	0.046	0.079	0.002	0.002	0.121	#13	0.206	Not Required	Pass
10	0.011	0.707	0.205	0.070	0.044	0.774	#13	0.082	Not Required	Pass
11	0.003	0.072	0.271	0.048	0.022	0.293	#21	0.102	Not Required	Pass
12	0.004	0.358	0.371	0.079	0.072	0.730	#13	0.036	Not Required	Pass
13	0.008	0.253	0.573	0.060	0.027	0.711	#21	0.306	Not Required	Pass
14	0.010	0.256	0.564	0.060	0.027	0.701	#21	0.204	Not Required	Pass
15	0.000	0.087	0.231	0.031	0.014	0.290	#21	Not Required	Not Required	Pass
16	0.000	0.087	0.231	0.031	0.014	0.290	#21	Not Required	Not Required	Pass
101	0.036	0.855	0.013	0.063	0.001	0.878	#13	0.419	Not Required	Pass
102	0.004	0.358	0.371	0.079	0.072	0.730	#13	0.036	Not Required	Pass
103	0.010	0.710	0.066	0.071	0.008	0.755	#13	0.046	Not Required	Pass
104	0.011	0.707	0.205	0.070	0.044	0.774	#13	0.082	Not Required	Pass
105	0.010	0.440	0.214	0.070	0.056	0.474	#13	0.076	Not Required	Pass
106	0.008	0.636	0.045	0.063	0.006	0.653	#13	0.046	Not Required	Pass
107	0.007	0.395	0.161	0.063	0.043	0.420	#13	0.076	Not Required	Pass
108	0.000	0.087	0.231	0.031	0.014	0.290	#21	Not Required	Not Required	Pass
109	0.021	0.046	0.079	0.002	0.002	0.121	#13	0.206	Not Required	Pass
110	0.007	0.633	0.162	0.063	0.035	0.705	#13	0.082	Not Required	Pass
111	0.000	0.087	0.231	0.031	0.014	0.290	#21	Not Required	Not Required	Pass
112	0.005	0.286	0.325	0.066	0.065	0.613	#13	0.036	Not Required	Pass
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115	0.008	0.579	0.308	0.048	0.022	0.761	#13	0.644	Not Required	Pass
116	0.002	0.581	0.307	0.048	0.022	0.762	#13	0.644	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
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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 8.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <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="652 1290 943 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.722</td> <td>11.466</td> </tr> <tr> <td>V_x (kip)</td> <td>-6.085</td> <td>-10.142</td> </tr> <tr> <td>V_z (kip)</td> <td>0.139</td> <td>0.218</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.464</td> <td>0.711</td> </tr> <tr> <td>M_z (kipft)</td> <td>75.172</td> <td>126.282</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ 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)	7.722	11.466	V_x (kip)	-6.085	-10.142	V_z (kip)	0.139	0.218	M_x (kipft)	0.464	0.711	M_z (kipft)	75.172	126.282	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>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:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-6.085 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.96895 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(75.172 \text{ kipft}) + ((-6.085 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.97 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.9215 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.139 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.022134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.464 \text{ kipft}) + ((0.139 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.073885 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0516 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9215 \text{ ft}), (2.0516 \text{ ft})]$$

$$L_{e,req} = 7.921 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.921 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.90526$$

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

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

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

$$q = \frac{(7.722 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.48262 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.48262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24131$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.96895 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 11.97 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.97 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.97 \text{ kipft/ft})) + (4 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0672 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.97 \text{ kipft/ft})) + (3 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (11.97 \text{ kipft/ft})) + (2 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.26038 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (11.97 \text{ kipft/ft})) + ((-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2117 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0672 \text{ ft})}{2}$$

$$p_a = 0.45504 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26038 \text{ kip/ft}^2)}{(0.45504 \text{ kip/ft}^2)}$$

$$Ratio = 0.57221$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2117 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.9232$$

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

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = 0.022134 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.073885 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.073885 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.022134 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.073885 \text{ kipft/ft})) + (4 \times (0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2971 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.073885 \text{ kipft/ft})) + (3 \times (0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.073885 \text{ kipft/ft})) + (2 \times (0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.012359 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.073885 \text{ kipft/ft})) + ((0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 0.026758 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2971 \text{ ft})}{2}$$

$$p_a = 0.47228 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.012359 \text{ kip/ft}^2)}{(0.47228 \text{ kip/ft}^2)}$$

$$Ratio = 0.026169$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

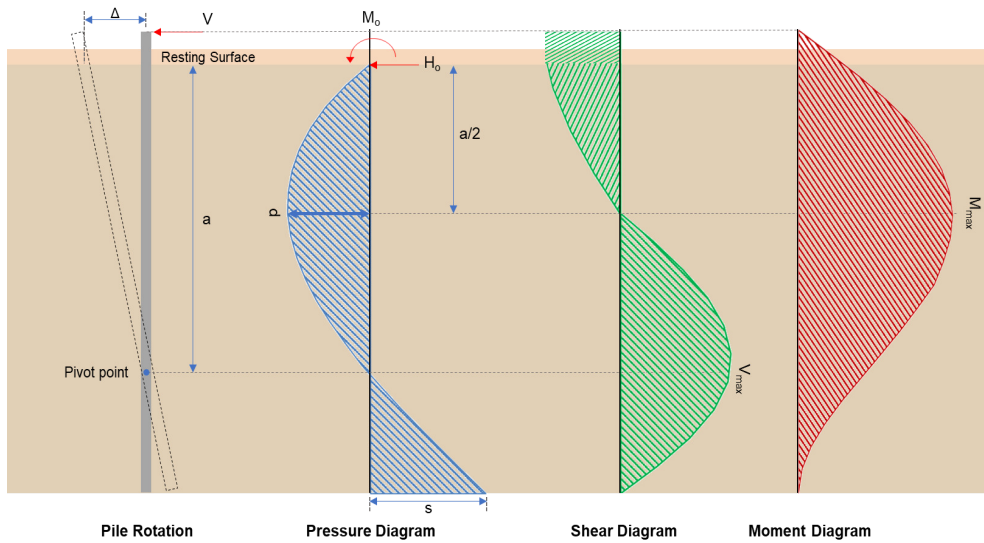
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.026758 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.020387$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-10.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(126.28 \text{ kipft}) + ((-10.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.109 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(20.109 \text{ kipft/ft})}{(-1.615 \text{ kip/ft})}$$

$$E = 12.451 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (20.109 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (20.109 \text{ kipft/ft})) + (4 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = \frac{(6 \times (20.109 \text{ kipft/ft})) + (4 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft}))}{(6 \times (20.109 \text{ kipft/ft})) + (4 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.066 \text{ ft}$$

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

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

$$V_{max} = ((-1.615 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.451 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.066 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.451 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.066 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 20.526 \text{ kip}$$

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

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

$$M_{max} = ((-1.615 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(12.451 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.451 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.451 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 84.681 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.218 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.034713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.711 \text{ kipft}) + ((0.218 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11322 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11322 \text{ kipft/ft})}{(0.034713 \text{ kip/ft})}$$

$$E = 3.2615 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11322 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.034713 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.11322 \text{ kipft/ft})) + (4 \times (0.034713 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.301 \text{ ft}$$

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

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

$$V_{max} = ((0.034713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2615 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.301 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2615 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.301 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.18452 \text{ kip}$$

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

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

$$M_{max} = ((0.034713 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(3.2615 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.301 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2615 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.301 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2615 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.301 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.6993 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.466 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.215 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.215 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.466 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0042861$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.466 \text{ kip} \rightarrow 11466 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11466 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.01 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.01 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.01 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((120.01 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.09 \text{ kip}$$

Considering x-direction:

V_{max} = 20.526 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(20.526 \text{ kip})}{(111.09 \text{ kip})}$$

$$Ratio = 0.18476$$

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

Considering z-direction:

$V_{max} = 0.18452 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.18452 \text{ kip})}{(111.09 \text{ kip})}$$

$$Ratio = 0.001661$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 84.681 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(84.681 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.33927$$

Status: **PASS**
Ratio: **0.340**

Considering z-direction:

$M_{max} = 0.6993 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.6993 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0028017$$

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

REFERENCES	CALCULATIONS	RESULTS
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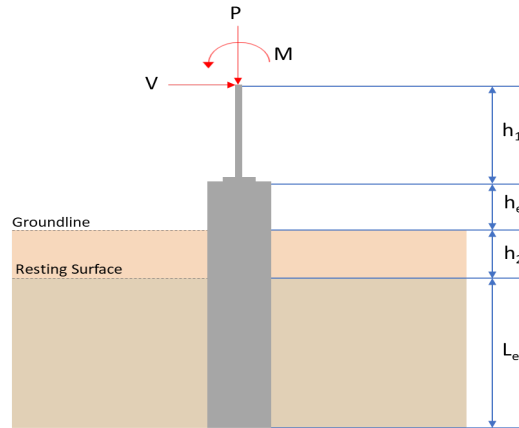
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8.75$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.722	11.466
V_x (kip)	-6.085	-10.142
V_z (kip)	-0.139	-0.218
M_x (kipft)	-0.465	-0.712
M_z (kipft)	75.172	126.284

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.085 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96895 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(75.172 \text{ kipft}) + ((-6.085 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.97 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.9215 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.139 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.022134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.465 \text{ kipft}) + ((-0.139 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.074045 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5664 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9215 \text{ ft}), (1.5664 \text{ ft})]$$

$$L_{e,req} = 7.921 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.921 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.90526$$

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

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

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

$$q = \frac{(7.722 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.48262 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.48262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24131$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.96895 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 11.97 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.97 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.97 \text{ kipft/ft})) + (4 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0672 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (11.97 \text{ kipft/ft})) + (3 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (11.97 \text{ kipft/ft})) + (2 \times (-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.26038 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (11.97 \text{ kipft/ft})) + ((-0.96895 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2117 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0672 \text{ ft})}{2}$$

$$p_a = 0.45504 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26038 \text{ kip/ft}^2)}{(0.45504 \text{ kip/ft}^2)}$$

$$Ratio = 0.57221$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.2117 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.9232$$

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

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = -0.022134 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.074045 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.074045 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.022134 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.074045 \text{ kipft/ft})) + (4 \times (-0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2967 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.074045 \text{ kipft/ft})) + (3 \times (-0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.074045 \text{ kipft/ft})) + (2 \times (-0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = -0.0048106 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.074045 \text{ kipft/ft})) + ((-0.022134 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = -0.0035721 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2967 \text{ ft})}{2}$$

$$p_a = 0.47226 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0048106 \text{ kip/ft}^2)}{(0.47226 \text{ kip/ft}^2)}$$

$$Ratio = -0.010186$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

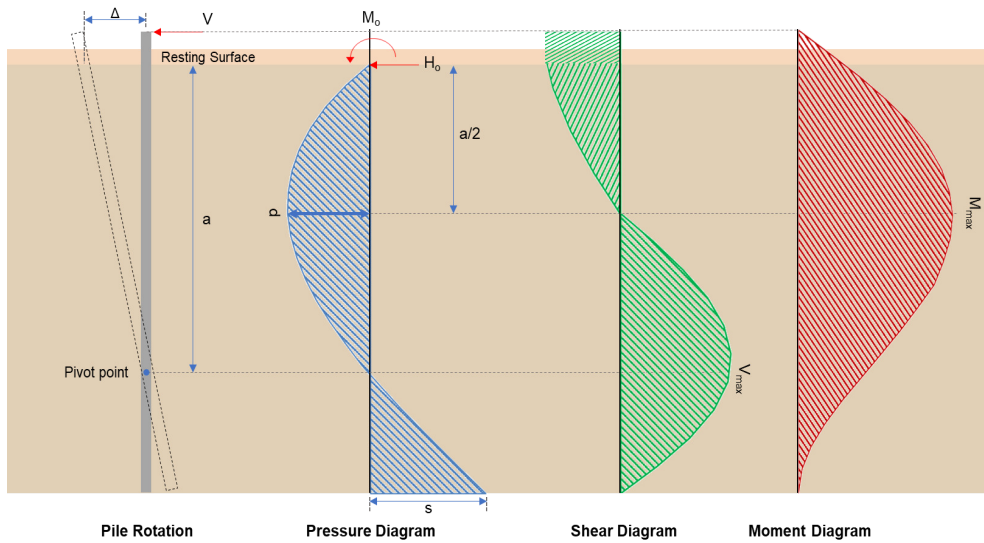
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0035721 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = -0.0027216$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-10.142 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(126.28 \text{ kipft}) + ((-10.142 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 20.109 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(20.109 \text{ kipft/ft})}{(-1.615 \text{ kip/ft})}$$

$$E = 12.452 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (20.109 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (20.109 \text{ kipft/ft})) + (4 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = \frac{(-1.615 \text{ kip/ft}) \times (8.75 \text{ ft})}{(6 \times (20.109 \text{ kipft/ft})) + (4 \times (-1.615 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.066 \text{ ft}$$

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

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

$$V_{max} = ((-1.615 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.452 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.066 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.452 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.066 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 20.526 \text{ kip}$$

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

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

$$M_{max} = ((-1.615 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(12.452 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.452 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.452 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.066 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 84.682 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.218 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.034713 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.712 \text{ kipft}) + ((-0.218 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11338 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11338 \text{ kipft/ft})}{(-0.034713 \text{ kip/ft})}$$

$$E = 3.2661 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11338 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.034713 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.11338 \text{ kipft/ft})) + (4 \times (-0.034713 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.3008 \text{ ft}$$

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

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

$$V_{max} = ((-0.034713 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2661 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.3008 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.2661 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.3008 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.18464 \text{ kip}$$

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

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

$$M_{max} = ((-0.034713 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(3.2661 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.3008 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.2661 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.3008 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2661 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.3008 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.69986 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{11.466 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.215 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.215 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(11.466 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0042861$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.466 \text{ kip} \rightarrow 11466 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(11466 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.01 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.01 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.01 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN} [V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN} [(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((120.01 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.09 \text{ kip}$$

Considering x-direction:

V_{max} = 20.526 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(20.526 \text{ kip})}{(111.09 \text{ kip})}$$

$$Ratio = 0.18477$$

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

Considering z-direction:

$V_{max} = 0.18464 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.18464 \text{ kip})}{(111.09 \text{ kip})}$$

$$Ratio = 0.0016621$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 84.682 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(84.682 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.33927$$

Status: **PASS**
Ratio: **0.340**

Considering z-direction:

$M_{max} = 0.69986 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.69986 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0028039$$

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