

Project Name: SunUp Ranch - 4x5 - V1Jb **Date:** Mon Dec 02 2024
Location: 9J3F+65, East Gull Lake, MN 56401, USA **Number of Modules:** 20
Unique ID: 2P-15-6TOP-XD-45-L-5Hx4W-GKH9 **Number of Poles:** 2
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	30.24 ft
Winter Tilt Angle	20
Front Edge Clearance	8 ft

MT Solar Bill of Materials (2P-15-6TOP-XD-45-L-5Hx4W-GKH9)

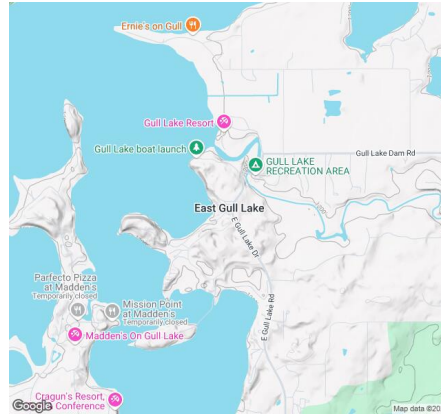
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	2
MTS-HF-XD	H-Frame Assembly-XD	2
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	2
MTS-CLAMP-ANGLE-4PK	Angle Clamp	4

Rail Bill of Materials

Part	Qty
Rails (223in)	8
Rail Attachment	32
Module Mid Clamp	32
Module End Clamp	16

Part	Qty
Ground Lug	4

Site Details:



Site Address: 9J3F+65, East Gull Lake, MN 56401, USA

Array Specification

Duty Classification:	XD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	5
Number of Columns:	4
Total Number of Modules:	20
Winter Tilt Angle:	20
Front Edge Clearance:	8
Total Array Height at Tilt:	14.43 ft
Total Frame Length:	30.00 ft
Frame Weight:	2114 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	30.24 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	11.22 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.50 ft Pile 2: 5.50 ft
Foundation Volume:	6.519 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	9J3F+65, East Gull Lake, MN 56401, USA
Wind Speed:	102 mph
Snow Load:	60 psf

Design Disclaimer

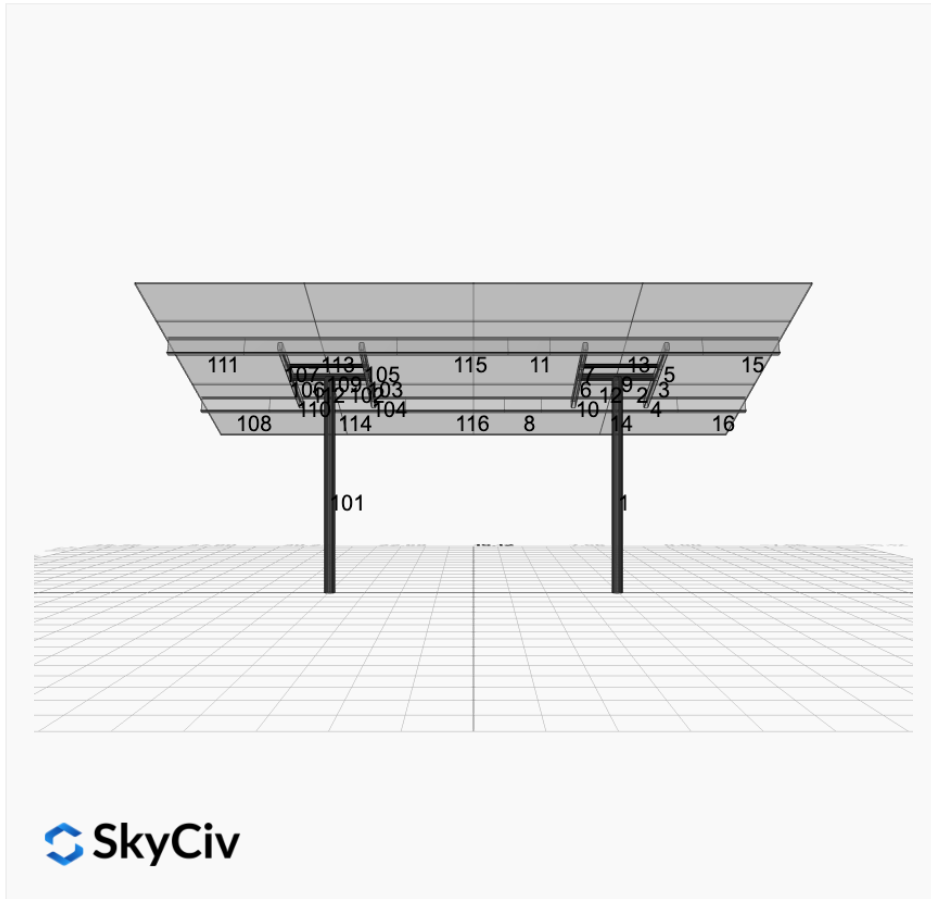
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

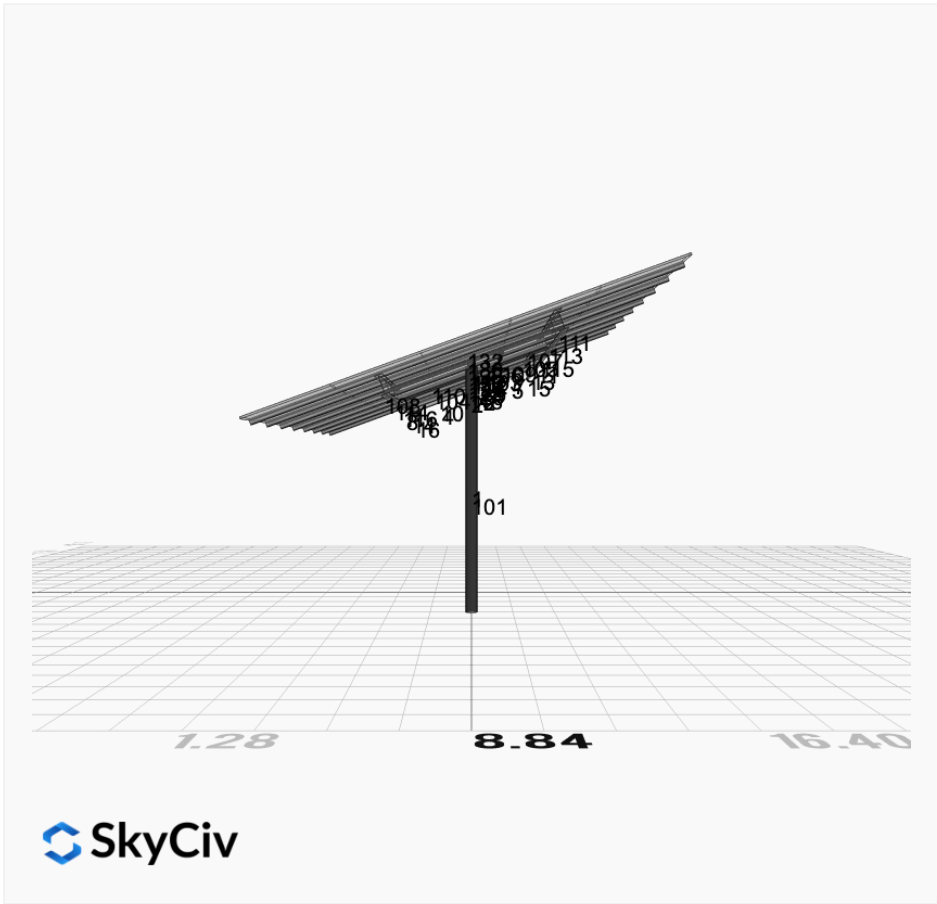
AutoDesigner Input

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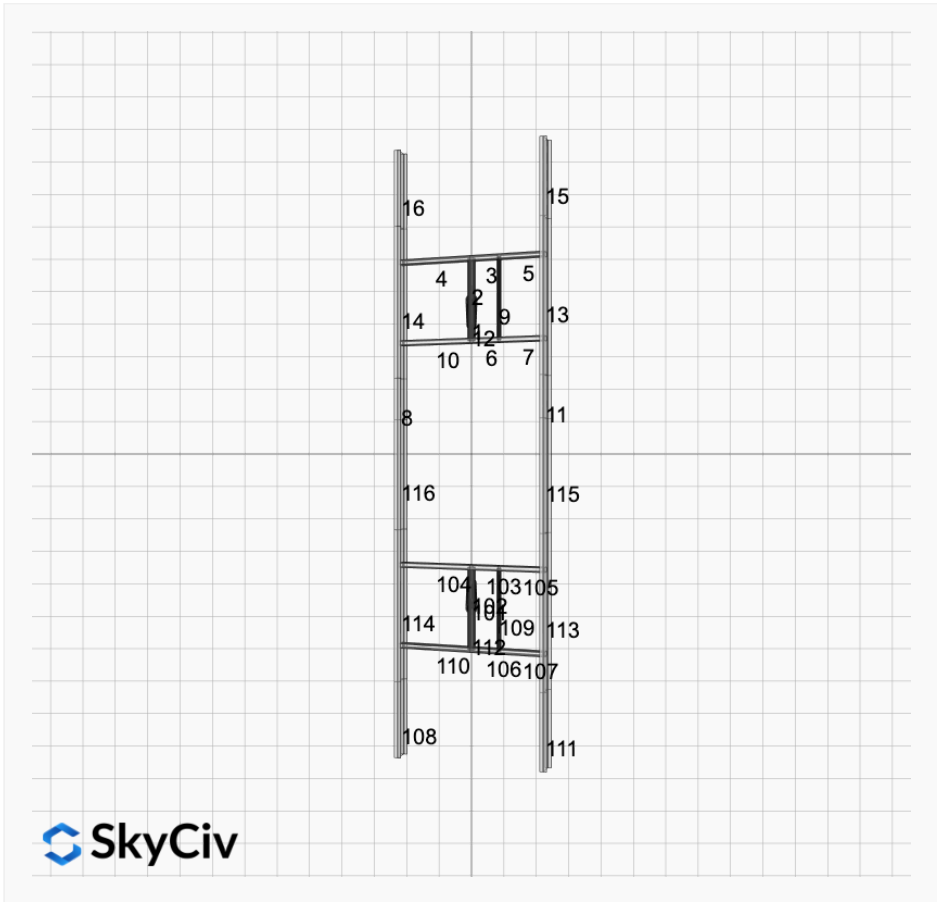
Design Notes:

- 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)

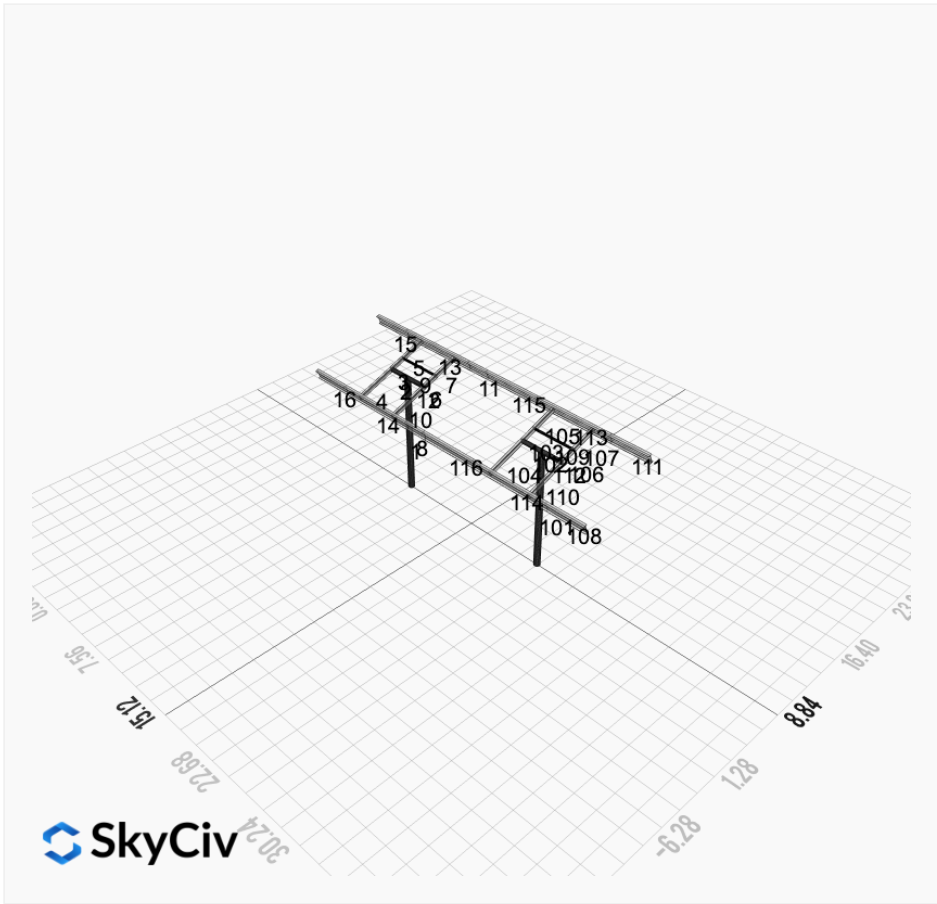




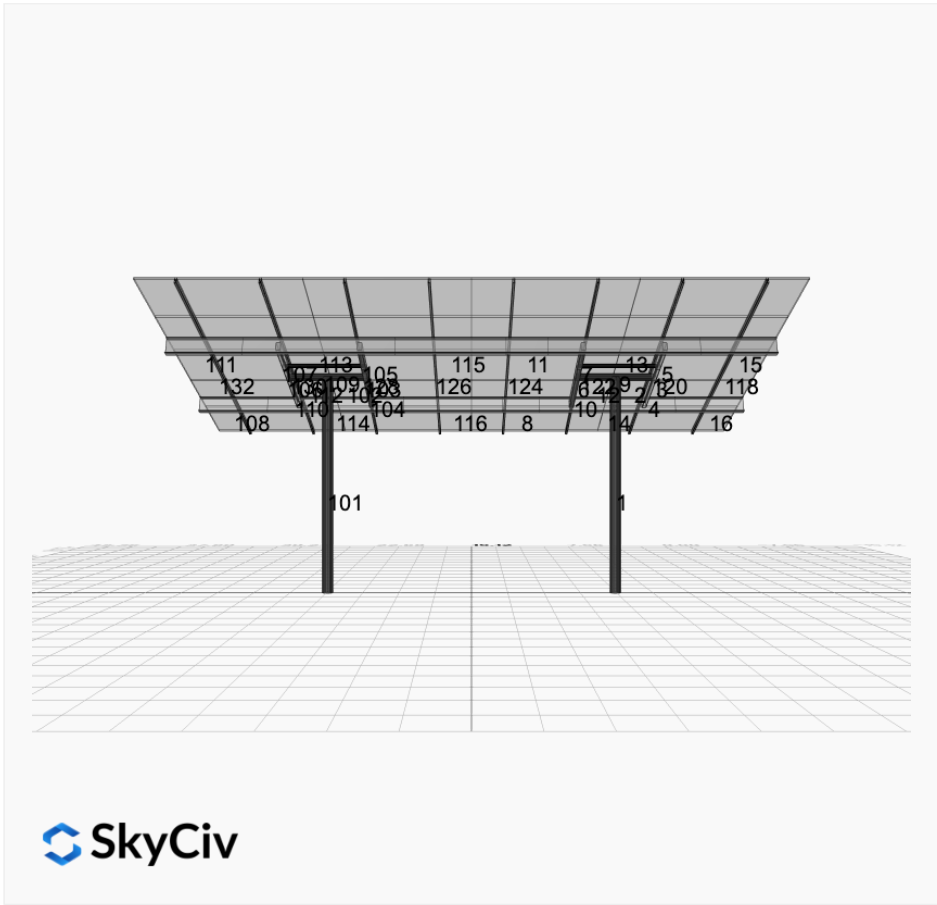
SkyCiv



SkyCiv



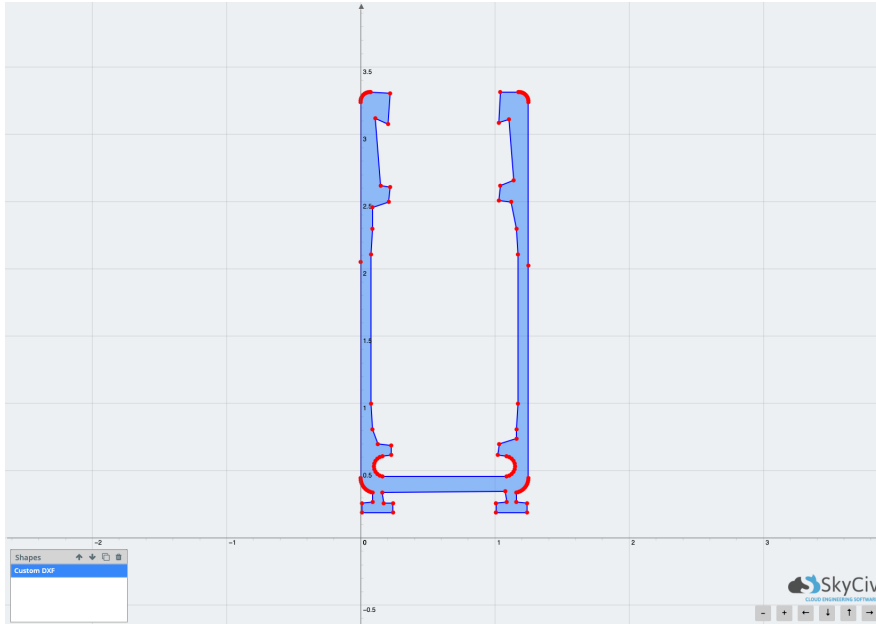
SkyCiv



SkyCiv

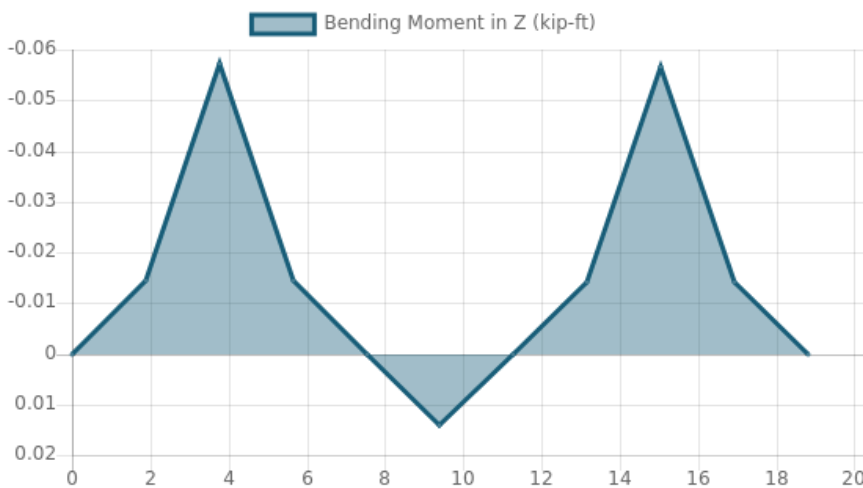
Rail Design Check

Rail Length: 18.8125 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.78 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.1172 kip/ft
Snow (Y): -0.0426 kip/ft
Wind uplift Case A: 0.0600 kip/ft
Wind uplift Case A: 0.0600 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0893 kip/ft

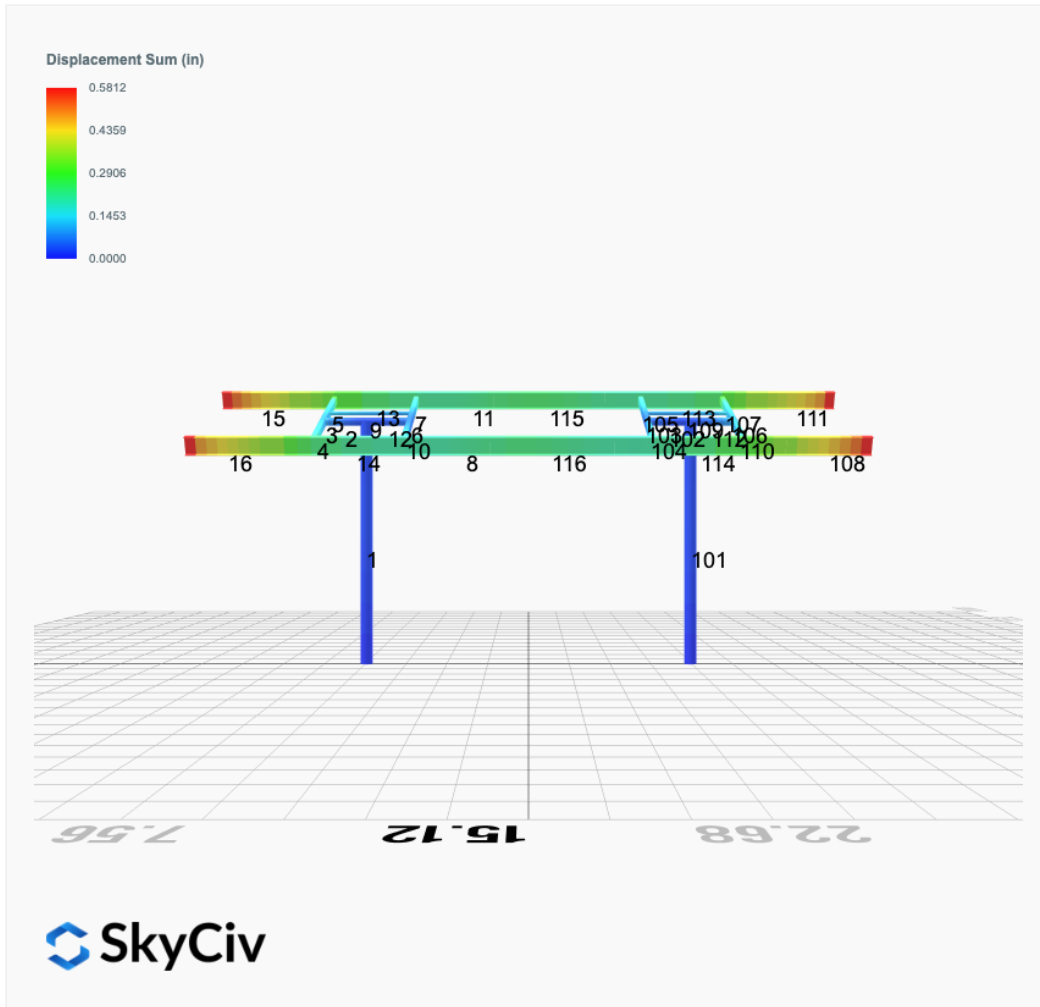


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	17.42569891	0.505	PASS
Material Yield	34.5	17.42569891	0.505	PASS
Material Strength	37	17.42569891	0.471	PASS

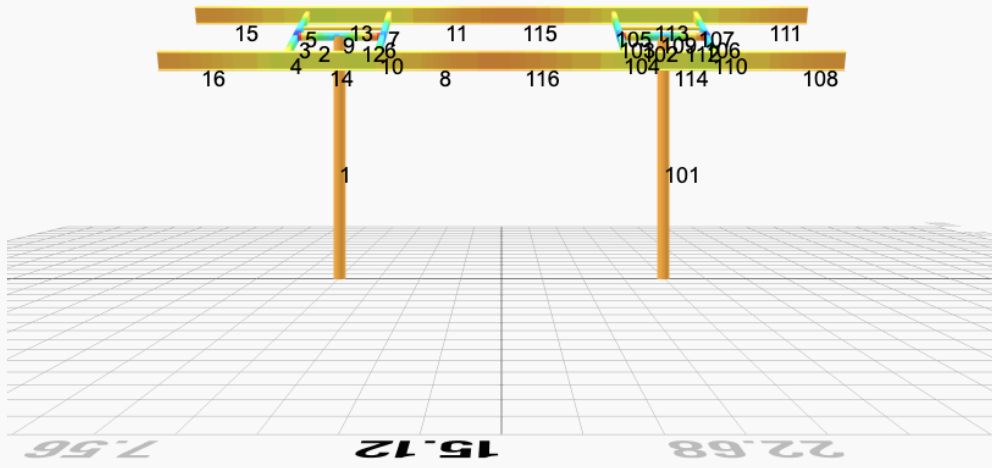
Member 1, ULS: 1. 1.4D



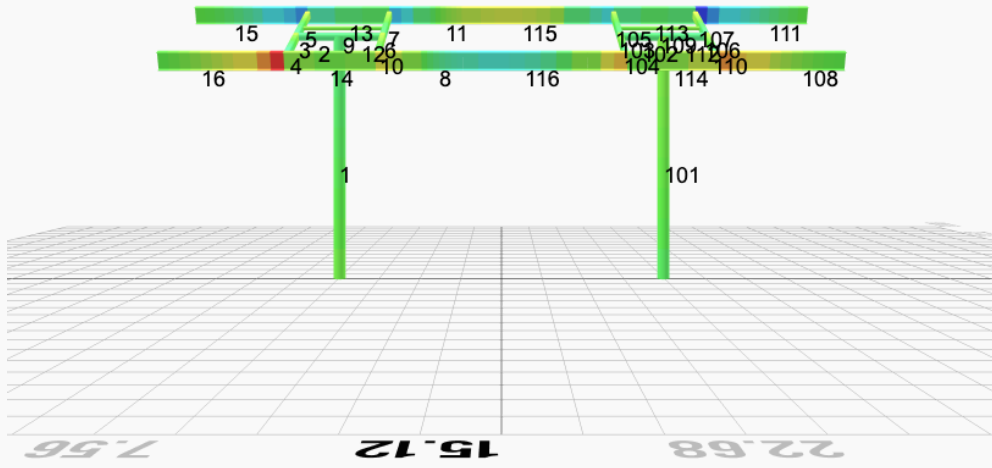
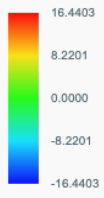
FEM Results (Envelope Worst Case for each member)



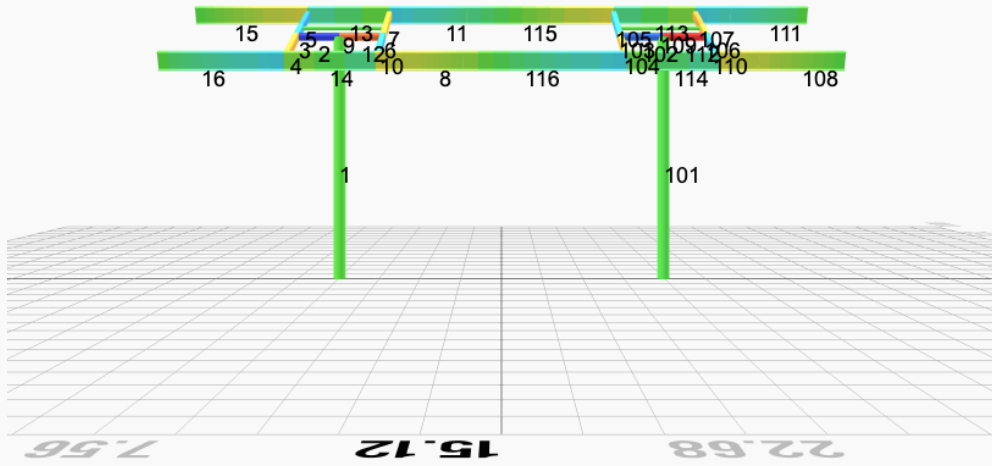
Top Bending Stress Z (ksi)



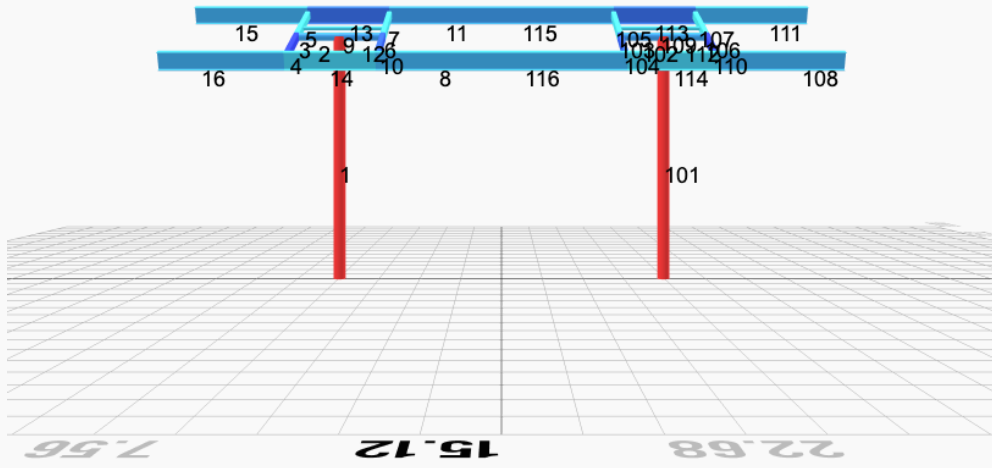
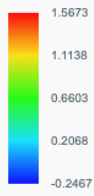
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (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	2.1028	-0.0259	-0.0924	0.0192	0.0317
ULS: 2. D + L	0.0000	2.1028	-0.0259	-0.0924	0.0192	0.0317
ULS: 3. D + (S or Lr or R)	-0.0000	10.8505	-0.1649	-0.5916	0.1221	0.0874
ULS: 3. D + (S or Lr or R)	0.0000	2.1028	-0.0259	-0.0924	0.0192	0.0317
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	8.6636	-0.1302	-0.4668	0.0964	0.0735
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1028	-0.0259	-0.0924	0.0192	0.0317
ULS: 5b. D + 0.7E	0.0000	2.1028	-0.0259	-0.0924	0.0192	0.0317
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	8.6636	-0.1302	-0.4668	0.0964	0.0735
ULS: 8. 0.6D + 0.7E	0.0000	1.2617	-0.0155	-0.0554	0.0115	0.0190
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0192	4.9032	-0.0751	-0.2680	0.0641	12.4715
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0192	4.9032	-0.0751	-0.2680	0.0641	12.4715
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8648	-0.2732	0.0158	0.0552	-0.0188	-8.7457
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7516	0.0380	0.0107	0.0373	-0.0142	-14.8211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	10.7639	-0.1671	-0.5985	0.1301	9.4033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	10.7639	-0.1671	-0.5985	0.1301	9.4033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	6.8816	-0.0989	-0.3561	0.0678	-6.5096
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	7.1149	-0.1027	-0.3695	0.0713	-11.0661
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	4.2031	-0.0628	-0.2241	0.0529	9.3616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	4.2031	-0.0628	-0.2241	0.0529	9.3616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	0.3208	0.0054	0.0183	-0.0093	-6.5514
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	0.5542	0.0016	0.0048	-0.0059	-11.1079
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0192	4.0620	-0.0648	-0.2310	0.0564	12.4589
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0192	4.0620	-0.0648	-0.2310	0.0564	12.4589
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8648	-1.1143	0.0261	0.0922	-0.0265	-8.7584
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7516	-0.8032	0.0211	0.0742	-0.0219	-14.8338

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	18.8533
Shear X	-1.6987
Shear Z	-0.2952
Moment X	-1.0665
Moment Y (Twist)	0.2252
Moment Z	26.3042

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	10.8505
Shear X	-1.0192
Shear Z	-0.1671
Moment X	-0.5985
Moment Y (Twist)	0.1301
Moment Z	14.8338

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	2.1028	0.0259	0.0924	-0.0192	0.0317
ULS: 2. D + L	-0.0000	2.1028	0.0259	0.0924	-0.0192	0.0317
ULS: 3. D + (S or Lr or R)	0.0000	10.8505	0.1649	0.5916	-0.1221	0.0874
ULS: 3. D + (S or Lr or R)	-0.0000	2.1028	0.0259	0.0924	-0.0192	0.0317
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	8.6636	0.1302	0.4668	-0.0964	0.0735

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.1028	0.0259	0.0924	-0.0192	0.0317
ULS: 5b. D + 0.7E	-0.0000	2.1028	0.0259	0.0924	-0.0192	0.0317
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	8.6636	0.1302	0.4668	-0.0964	0.0735
ULS: 8. 0.6D + 0.7E	-0.0000	1.2617	0.0155	0.0554	-0.0115	0.0190
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0192	4.9032	0.0751	0.2680	-0.0641	12.4715
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0192	4.9032	0.0751	0.2680	-0.0641	12.4715
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8648	-0.2732	-0.0158	-0.0552	0.0188	-8.7457
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.7516	0.0380	-0.0107	-0.0373	0.0142	-14.8211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	10.7639	0.1671	0.5985	-0.1301	9.4033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	10.7639	0.1671	0.5985	-0.1301	9.4033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	6.8816	0.0989	0.3561	-0.0678	-6.5096
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	7.1150	0.1027	0.3695	-0.0713	-11.0661
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7644	4.2031	0.0628	0.2241	-0.0529	9.3616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7644	4.2031	0.0628	0.2241	-0.0529	9.3616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6486	0.3208	-0.0054	-0.0183	0.0093	-6.5514
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5637	0.5542	-0.0016	-0.0048	0.0059	-11.1079
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0192	4.0620	0.0648	0.2310	-0.0564	12.4589
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0192	4.0620	0.0648	0.2310	-0.0564	12.4589
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8648	-1.1143	-0.0261	-0.0922	0.0265	-8.7584
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7516	-0.8032	-0.0211	-0.0742	0.0219	-14.8338

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	18.8533
Shear X	-1.6987
Shear Z	0.2952
Moment X	1.0666
Moment Y (Twist)	0.2253
Moment Z	26.3049

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	10.8505
Shear X	-1.0192
Shear Z	0.1671
Moment X	0.5985
Moment Y (Twist)	0.1301
Moment Z	14.8338

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: SunUp Ranch - 4x5 - V1Jb
 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)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
7	6in Pipe Sch 40	6.63	0.28					

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

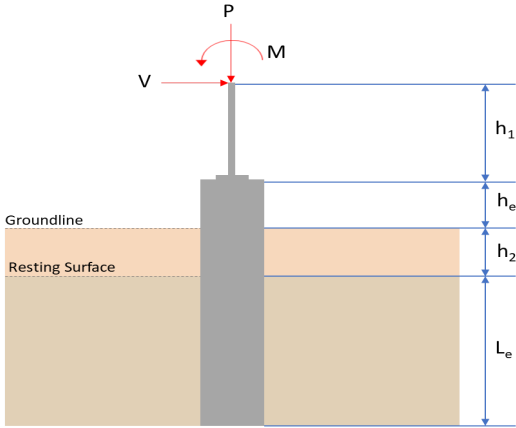
Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

4	0.010	0.046	0.153	0.085	0.037	0.569	#21	0.076	Not Required	Pass
5	0.010	0.537	0.147	0.085	0.037	0.569	#21	0.076	Not Required	Pass
6	0.009	0.792	0.051	0.078	0.009	0.845	#21	0.046	Not Required	Pass
7	0.009	0.492	0.097	0.078	0.025	0.518	#21	0.076	Not Required	Pass
8	0.001	0.064	0.114	0.039	0.015	0.119	#24	0.102	Not Required	Pass
9	0.007	0.090	0.053	0.002	0.002	0.146	#21	0.206	Not Required	Pass
10	0.008	0.774	0.108	0.077	0.026	0.872	#21	0.082	Not Required	Pass
11	0.002	0.064	0.115	0.039	0.015	0.121	#21	0.068	Not Required	Pass
12	0.002	0.540	0.075	0.117	0.011	0.616	#21	0.054	Not Required	Pass
13	0.007	0.289	0.426	0.057	0.022	0.695	#21	0.204	Not Required	Pass
14	0.009	0.290	0.426	0.056	0.022	0.691	#21	0.204	Not Required	Pass
15	0.000	0.088	0.198	0.039	0.015	0.286	#21	Not Required	Not Required	Pass
16	0.000	0.087	0.198	0.039	0.015	0.284	#21	Not Required	Not Required	Pass
101	0.237	0.622	0.053	0.023	0.004	0.647	#16	0.629	Not Required	Pass
102	0.002	0.540	0.075	0.117	0.011	0.616	#21	0.054	Not Required	Pass
103	0.009	0.792	0.051	0.078	0.009	0.845	#21	0.046	Not Required	Pass
104	0.008	0.774	0.108	0.077	0.026	0.872	#21	0.082	Not Required	Pass
105	0.009	0.492	0.097	0.078	0.025	0.518	#21	0.076	Not Required	Pass
106	0.010	0.866	0.082	0.087	0.021	0.952	#21	0.046	Not Required	Warn
107	0.010	0.537	0.147	0.085	0.037	0.569	#21	0.076	Not Required	Pass
108	0.000	0.087	0.198	0.039	0.015	0.284	#21	Not Required	Not Required	Pass
109	0.007	0.090	0.053	0.002	0.002	0.146	#21	0.206	Not Required	Pass
110	0.010	0.848	0.135	0.085	0.029	0.935	#21	0.082	Not Required	Pass
111	0.000	0.088	0.198	0.039	0.015	0.286	#21	Not Required	Not Required	Pass
112	0.001	0.622	0.083	0.130	0.012	0.706	#21	0.054	Not Required	Pass
113	0.007	0.289	0.426	0.057	0.022	0.695	#21	0.204	Not Required	Pass
114	0.009	0.290	0.426	0.056	0.022	0.691	#21	0.306	Not Required	Pass
115	0.002	0.064	0.153	0.039	0.015	0.177	#21	0.181	Not Required	Pass
116	0.001	0.064	0.153	0.039	0.015	0.175	#21	0.271	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 $L = 5.5$ 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="655 1290 940 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.851</td> <td>18.853</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.019</td> <td>-1.699</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.167</td> <td>-0.295</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.598</td> <td>-1.066</td> </tr> <tr> <td>M_z (kipft)</td> <td>14.834</td> <td>26.304</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)	10.851	18.853	V_x (kip)	-1.019	-1.699	V_z (kip)	-0.167	-0.295	M_x (kipft)	-0.598	-1.066	M_z (kipft)	14.834	26.304	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.019 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.16226 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.834 \text{ kipft}) + ((-1.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.3621 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.175 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.167 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.598 \text{ kipft}) + ((-0.167 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.095223 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.6993 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.175 \text{ ft}), (1.6993 \text{ ft})]$$

$$L_{e,req} = 5.175 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.175 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.94091$$

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

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

$$q = \frac{(10.851 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.67819 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.67819 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.33909$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16226 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.3621 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3621 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.3621 \text{ kipft/ft})) + (4 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7589 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3621 \text{ kipft/ft})) + (3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.3621 \text{ kipft/ft})) + (2 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.21442 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.3621 \text{ kipft/ft})) + ((-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.76002 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7589 \text{ ft})}{2}$$

$$p_a = 0.28192 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21442 \text{ kip/ft}^2)}{(0.28192 \text{ kip/ft}^2)}$$

$$Ratio = 0.76057$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.76002 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.92124$$

Status: **PASS**
Ratio: **0.760**

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

Considering z-direction:

$H_o = -0.026592 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.095223 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.095223 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.095223 \text{ kipft/ft})) + (4 \times (-0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8985 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.095223 \text{ kipft/ft})) + (3 \times (-0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.095223 \text{ kipft/ft})) + (2 \times (-0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.012132 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.095223 \text{ kipft/ft})) + ((-0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.0087645 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8985 \text{ ft})}{2}$$

$$p_a = 0.29239 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.012132 \text{ kip/ft}^2)}{(0.29239 \text{ kip/ft}^2)}$$

$$Ratio = -0.041491$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0087645 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.010624$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.699 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27054 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.304 \text{ kipft}) + ((-1.699 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1885 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1885 \text{ kipft/ft})}{(-0.27054 \text{ kip/ft})}$$

$$E = 15.482 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1885 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.1885 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = \frac{(6 \times (4.1885 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}{(6 \times (4.1885 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7544 \text{ ft}$$

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

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

$$V_{max} = ((-0.27054 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.482 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.482 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.1084 \text{ kip}$$

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

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

$$M_{max} = ((-0.27054 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(15.482 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7544 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.482 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7544 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (15.482 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7544 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 16.255 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.295 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046975 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.066 \text{ kipft}) + ((-0.295 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16975 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16975 \text{ kipft/ft})}{(-0.046975 \text{ kip/ft})}$$

$$E = 3.6136 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16975 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.046975 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.16975 \text{ kipft/ft})) + (4 \times (-0.046975 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8975 \text{ ft}$$

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

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

$$V_{max} = ((-0.046975 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6136 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.8975 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.6136 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.8975 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.34314 \text{ kip}$$

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

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

$$M_{max} = ((-0.046975 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(3.6136 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8975 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.6136 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.8975 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6136 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.8975 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85111 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(18.853 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.97 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.97 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(18.853 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0070474$	<p>Status: PASS Ratio: 0.010</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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 18.853 \text{ kip} \rightarrow 18853 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18853 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (121 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((121 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.73 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.1084 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.1084 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.054671$$

Considering z-direction:

$V_{max} = 0.34314 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.34314 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.0030712$$

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

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

$\phi M_{n,2}$

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

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

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

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:

$M_{max} = 16.255 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(16.255 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.065123$$

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

Considering z-direction:

$M_{max} = 0.85111 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.85111 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0034099$$

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

REFERENCES	CALCULATIONS	RESULTS
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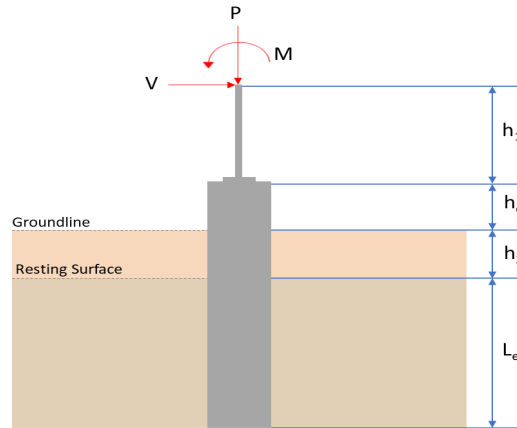
SkyCiv Foundation Design

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

$L = 5.5$ 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)	10.851	18.853
V_x (kip)	-1.019	-1.699
V_z (kip)	0.167	0.295
M_x (kipft)	0.598	1.067
M_z (kipft)	14.834	26.305

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,

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

$$H_o = \frac{(-1.019 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16226 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.834 \text{ kipft}) + ((-1.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.3621 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.175 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.167 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.026592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.598 \text{ kipft}) + ((0.167 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.095223 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.2364 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.175 \text{ ft}), (2.2364 \text{ ft})]$$

$$L_{e,req} = 5.175 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.175 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.94091$$

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

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

$$q = \frac{(10.851 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.67819 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.67819 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.33909$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.16226 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.3621 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3621 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.3621 \text{ kipft/ft})) + (4 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7589 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3621 \text{ kipft/ft})) + (3 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.3621 \text{ kipft/ft})) + (2 \times (-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.21442 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.3621 \text{ kipft/ft})) + ((-0.16226 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.76002 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7589 \text{ ft})}{2}$$

$$p_a = 0.28192 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.21442 \text{ kip/ft}^2)}{(0.28192 \text{ kip/ft}^2)}$$

$$Ratio = 0.76057$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.76002 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.92124$$

Status: **PASS**
Ratio: **0.760**

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

Considering z-direction:

$H_o = 0.026592 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.095223 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.095223 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.026592 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.095223 \text{ kipft/ft})) + (4 \times (0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8985 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.095223 \text{ kipft/ft})) + (3 \times (0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 [(3 \times (0.095223 \text{ kipft/ft})) + (2 \times (0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.02881 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.095223 \text{ kipft/ft})) + ((0.026592 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.066784 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8985 \text{ ft})}{2}$$

$$p_a = 0.29239 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.02881 \text{ kip/ft}^2)}{(0.29239 \text{ kip/ft}^2)}$$

$$Ratio = 0.098532$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.066784 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.080951$$

Status: **PASS**
Ratio: **0.100**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.699 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.27054 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.305 \text{ kipft}) + ((-1.699 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1887 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.1887 \text{ kipft/ft})}{(-0.27054 \text{ kip/ft})}$$

$$E = 15.483 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1887 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.1887 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = \frac{(-0.27054 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.1887 \text{ kipft/ft})) + (4 \times (-0.27054 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7544 \text{ ft}$$

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

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

$$V_{max} = ((-0.27054 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.483 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.483 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7544 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.1087 \text{ kip}$$

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

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

$$M_{max} = ((-0.27054 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(15.483 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7544 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.483 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7544 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (15.483 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7544 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 16.255 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.295 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.046975 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.067 \text{ kipft}) + ((0.295 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1699 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1699 \text{ kipft/ft})}{(0.046975 \text{ kip/ft})}$$

$$E = 3.6169 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1699 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.046975 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.1699 \text{ kipft/ft})) + (4 \times (0.046975 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.8974 \text{ ft}$$

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

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

$$V_{max} = ((0.046975 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.6169 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.8974 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.6169 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.8974 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.34335 \text{ kip}$$

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

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

$$M_{max} = ((0.046975 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(3.6169 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.8974 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.6169 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.8974 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.6169 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.8974 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.85167 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(18.853 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.97 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.97 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(18.853 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0070474$</p>	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 18.853 \text{ kip} \rightarrow 18853 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(18853 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (121 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((121 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.73 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.1087 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.1087 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.054673$$

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

Considering z-direction:

$V_{max} = 0.34335 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.34335 \text{ kip})}{(111.73 \text{ kip})}$$

$$Ratio = 0.003073$$

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 $\phi M_{n,2}$

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

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

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

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:

$M_{max} = 16.255 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.255 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.065125$$

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

Considering z-direction:

$M_{max} = 0.85167 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.85167 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0034121$$

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