

Your Project Calculations

Project Name: MTSOLAR_8J3EEK4CBB9I-JB-RevAstruts

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_8J3EEK4CBB9I-JB-RevAstruts&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=VfTOvrwWsVH05NVixjgbgPNMz8Mw6UO7rtly5XdN5MEsG362mgt7UyQ4w3MFoutc



Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-72-L-4Hx3W-7238
Duty Classification:	SD
Module Width:	41.50 in
Module Length:	82.13in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	55
Front Edge Clearance:	7
Total Array Height at Tilt:	18.40 ft
Total Frame Length:	19.50 ft
Frame Weight:	816 lbs
Array Dimensions N/S:	14.00 ft
Array Dimensions E/W:	20.78 ft
Rail Length:	168.00 in
Rail Spacing:	3.42 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.73 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.50 ft
Foundation Volume:	3.259 y ³
Foundation Result:	PASSED
Mount Twist:	0.001556 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sandy_gravel
Site Location:	F24F+99, Cascade, ID 83611, USA
Wind Speed:	95 mph
Snow Load:	96.2 psf
Design Uplift Pressure:	0.011432 ksf
Design Downforce Pressure:	-0.011432 ksf
Design Snow Pressure:	0.015868 ksf



Design Disclaimer

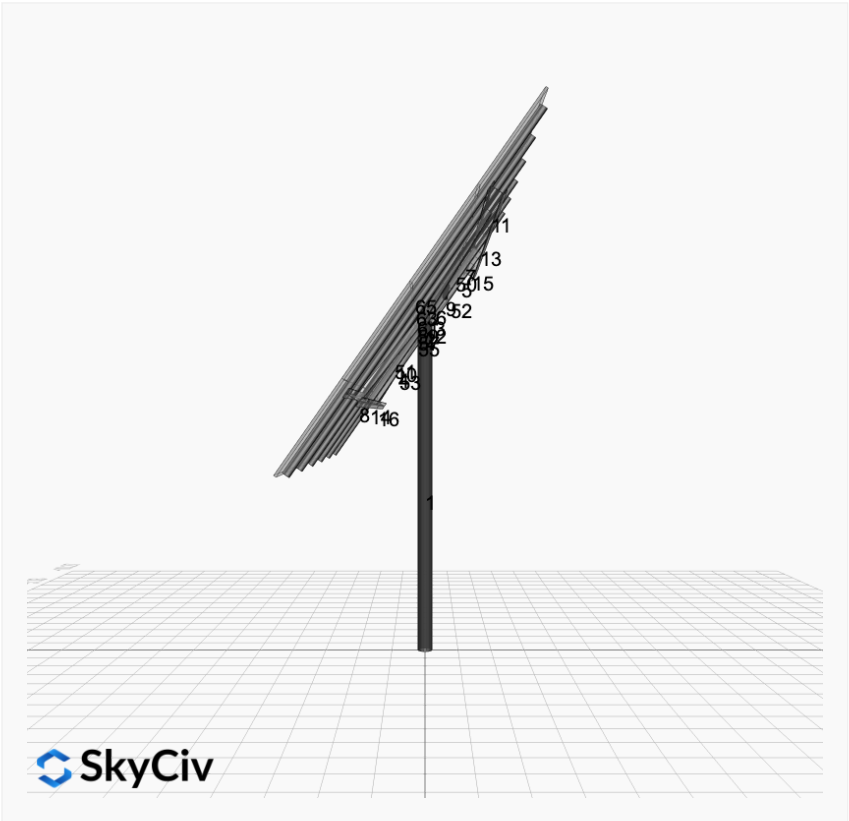
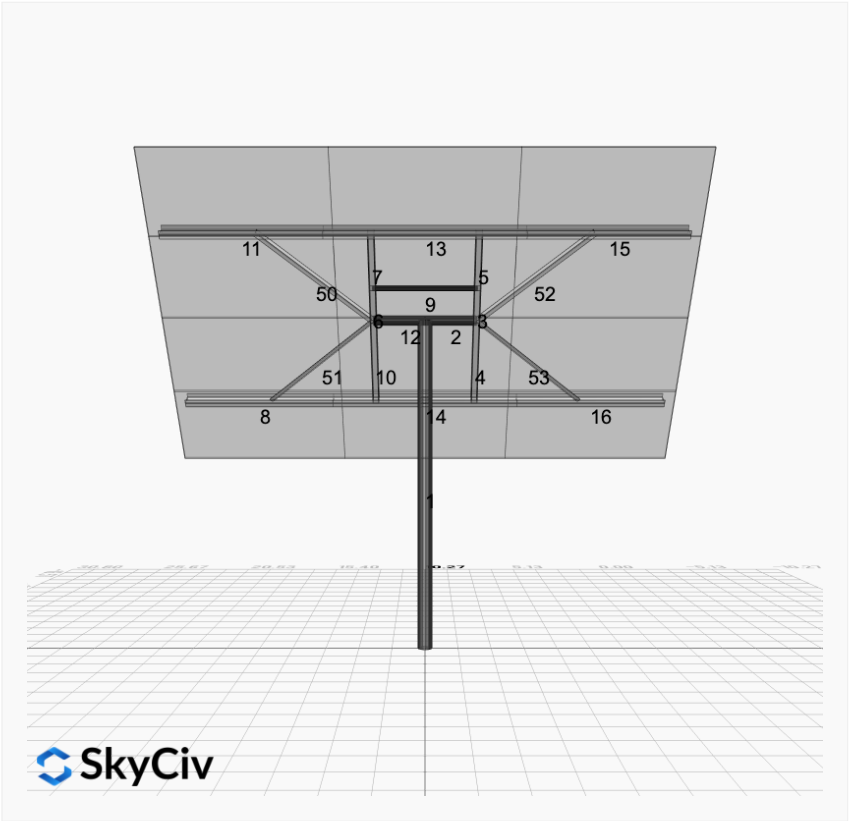
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

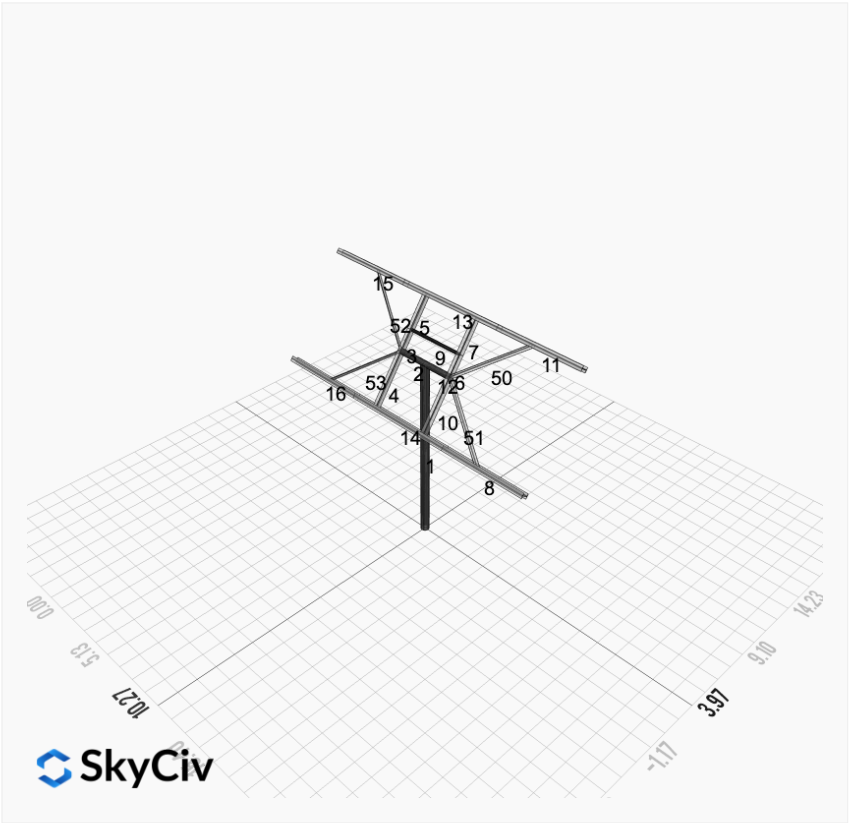
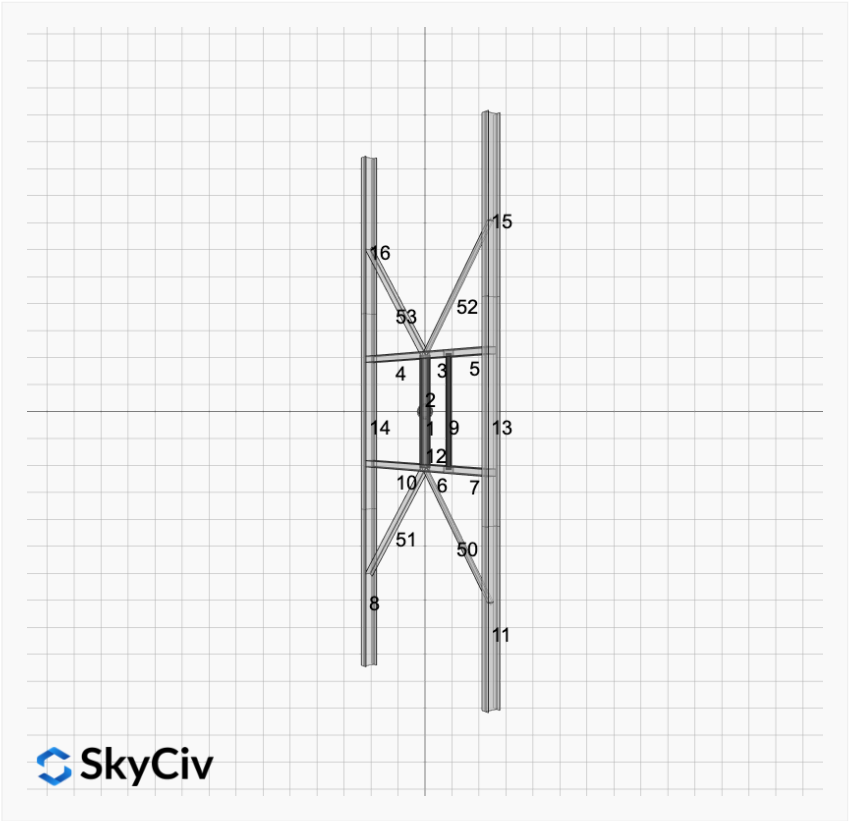
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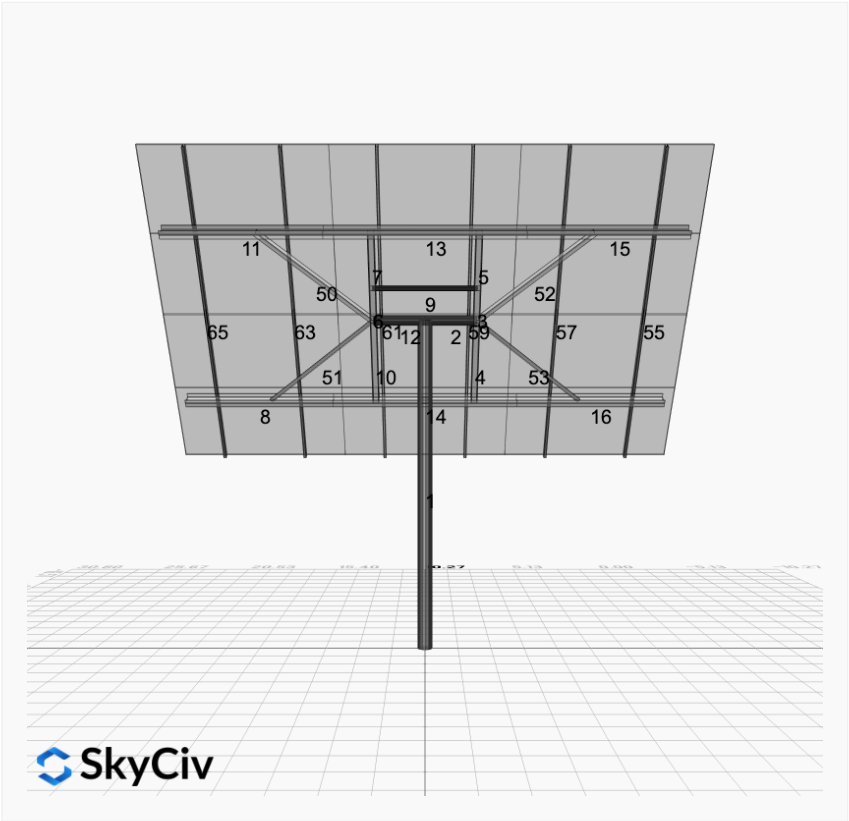
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Design Notes:

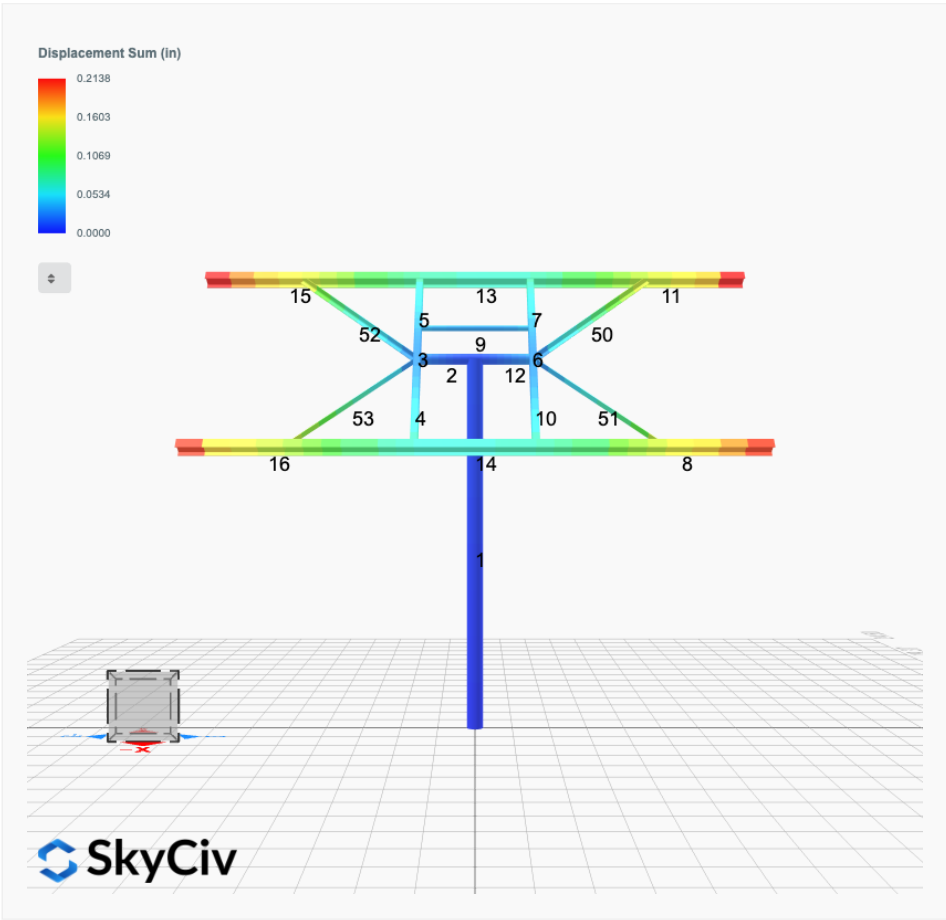
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles

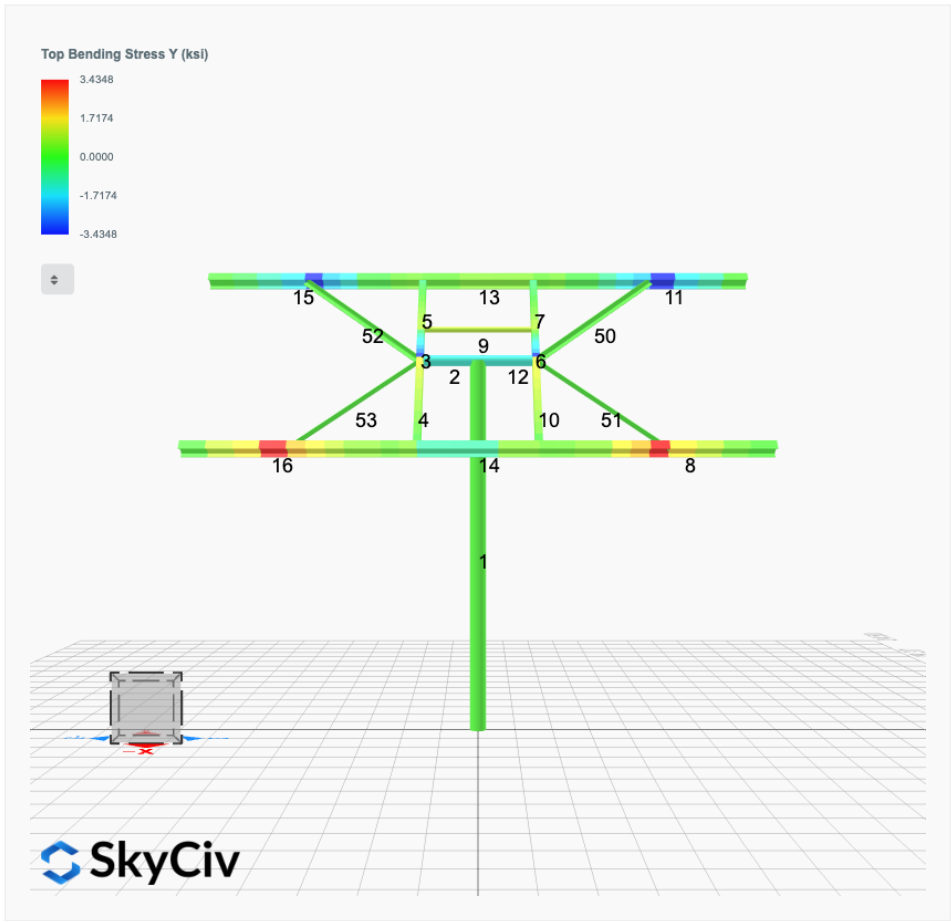
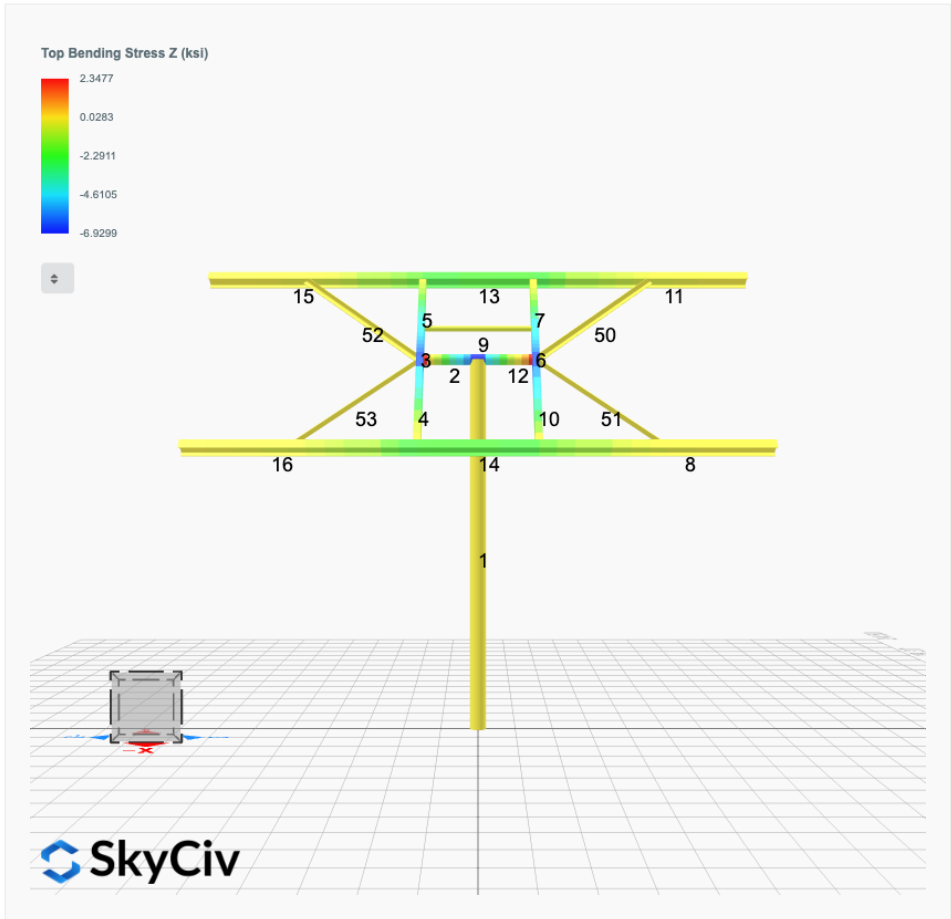


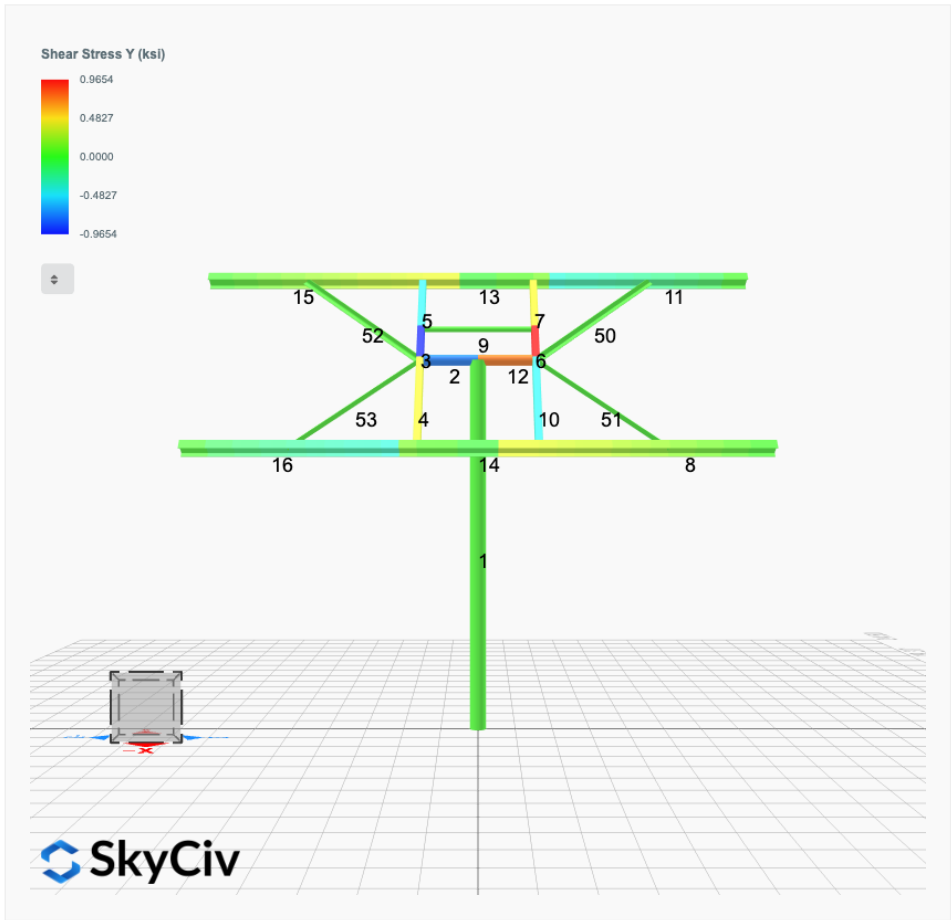


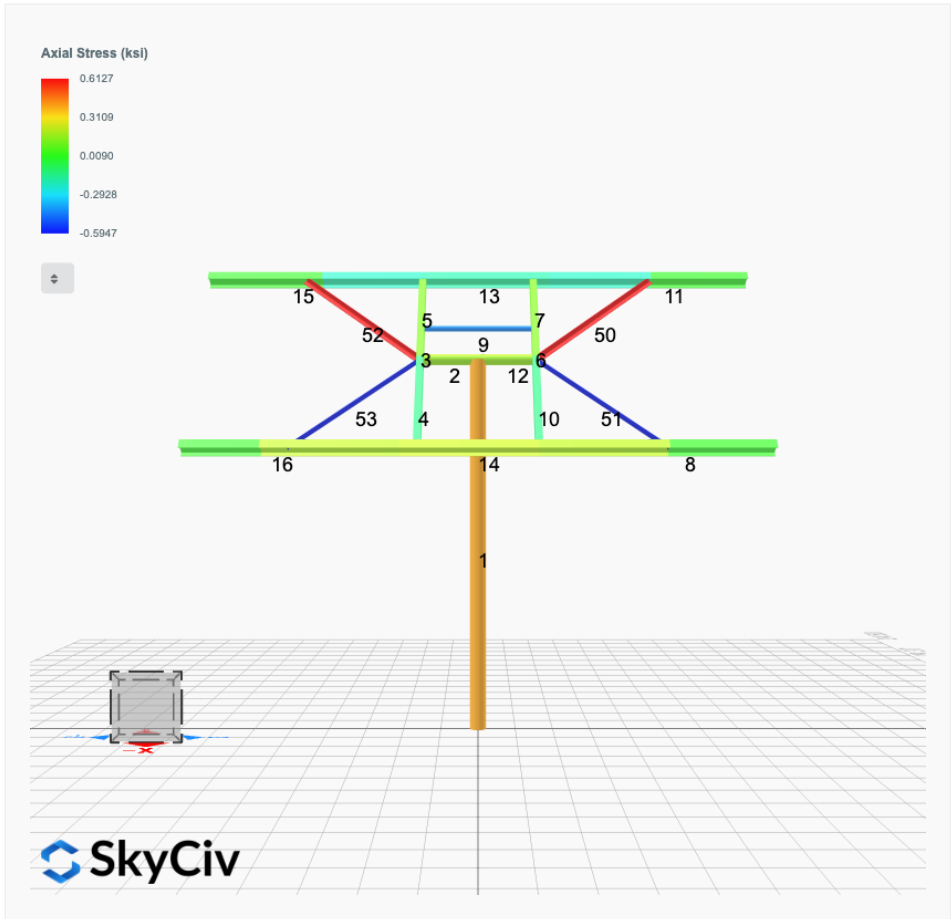


FEM Results (Envelope Worst Case for each member)









Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 2. D + L	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 3. D + (S or Lr or R)	-0.0000	4.6449	0.0000	-0.0000	-0.0000	0.0359
ULS: 3. D + (S or Lr or R)	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.0237	0.0000	-0.0000	-0.0000	0.0314
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 5b. D + 0.7E	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.0237	0.0000	-0.0000	-0.0000	0.0314
ULS: 8. 0.6D + 0.7E	-0.0000	1.2961	0.0000	-0.0000	-0.0000	0.0107
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6348	3.3050	0.0000	-0.0000	-0.0000	21.2229
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.6348	1.0155	0.0000	-0.0000	-0.0000	-20.4295
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2261	4.8823	0.0000	-0.0000	-0.0000	15.9353
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	4.0237	0.0000	-0.0000	-0.0000	0.0314
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2261	3.1652	0.0000	-0.0000	-0.0000	-15.3040
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	4.0237	0.0000	-0.0000	-0.0000	0.0314
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2261	3.0188	0.0000	-0.0000	-0.0000	15.9216
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2261	1.3017	0.0000	-0.0000	-0.0000	-15.3177
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.1602	0.0000	-0.0000	-0.0000	0.0178
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6348	2.4409	0.0000	-0.0000	-0.0000	21.2158
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.2961	0.0000	-0.0000	-0.0000	0.0107
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.6348	0.1514	0.0000	-0.0000	-0.0000	-20.4366
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.2961	0.0000	-0.0000	-0.0000	0.0107

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	7.5217
Shear X	-2.7247
Shear Z	0.0000
Moment X	-0.0026
Moment Y (Twist)	0.0016
Moment Z	36.8442

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	4.8823
Shear X	-1.6348
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	21.2229

Project Details

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

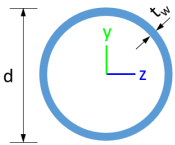
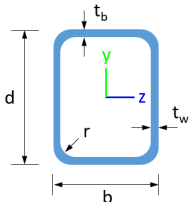
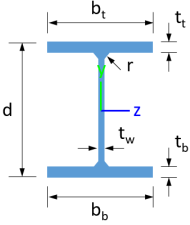
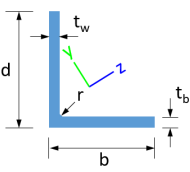
User Name: sales@mtsolar.us
Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions								
								
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
7	6in Pipe Sch 40	6.63	0.28					
								
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		
								
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25
								

[illegible]

Member Design Capacity

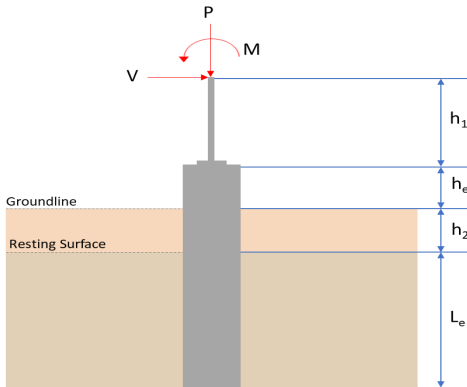
Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	61.74	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	76.00	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	76.00	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	17.96	6.45	30.09	45.74
14	120.60	98.23	17.96	6.45	30.09	45.74
15	120.60	76.00	23.36	6.45	30.09	45.74
16	120.60	76.00	23.36	6.45	30.09	45.74
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.122	0.871	0.000	0.036	0.000	0.918	#13	0.715	Not Required	Pass
2	0.007	0.345	0.188	0.084	0.032	0.480	#21	0.052	Not Required	Pass
3	0.005	0.462	0.282	0.046	0.066	0.739	#21	0.044	Not Required	Pass
4	0.005	0.457	0.105	0.046	0.008	0.554	#21	0.078	Not Required	Pass
5	0.005	0.286	0.055	0.046	0.011	0.339	#21	0.073	Not Required	Pass
6	0.005	0.462	0.282	0.046	0.066	0.739	#21	0.044	Not Required	Pass
7	0.005	0.286	0.055	0.046	0.011	0.339	#21	0.073	Not Required	Pass
8	0.013	0.106	0.120	0.027	0.010	0.161	#21	0.397	Not Required	Pass
9	0.027	0.035	0.053	0.001	0.000	0.098	#21	0.132	Not Required	Pass
10	0.005	0.457	0.105	0.046	0.008	0.554	#21	0.078	Not Required	Pass
11	0.009	0.106	0.120	0.028	0.010	0.159	#21	0.265	Not Required	Pass
12	0.007	0.345	0.188	0.084	0.032	0.480	#21	0.052	Not Required	Pass
13	0.009	0.232	0.023	0.035	0.006	0.255	#21	0.177	Not Required	Pass
14	0.012	0.235	0.026	0.035	0.006	0.248	#21	0.265	Not Required	Pass
15	0.009	0.106	0.120	0.028	0.010	0.159	#21	0.265	Not Required	Pass
16	0.013	0.106	0.120	0.027	0.010	0.161	#21	0.265	Not Required	Pass
50	0.163	0.009	0.006	0.002	0.002	0.094	#21	0.783	Not Required	Pass
51	0.032	0.003	0.015	0.002	0.002	0.030	#21	0.522	Not Required	Pass
52	0.163	0.009	0.006	0.002	0.002	0.094	#23	0.783	Not Required	Pass
53	0.032	0.003	0.015	0.002	0.002	0.030	#23	0.522	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																										
	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 h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sandy gravel and/or gravel</td><td>3000.000</td><td>200.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>4.882</td><td>7.522</td></tr><tr><td>Vx (kip)</td><td>-1.635</td><td>-2.725</td></tr><tr><td>Vz (kip)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mx (kipft)</td><td>0.000</td><td>-0.003</td></tr><tr><td>Mz (kipft)</td><td>21.223</td><td>36.844</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	4.882	7.522	Vx (kip)	-1.635	-2.725	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	-0.003	Mz (kipft)	21.223	36.844	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sandy gravel and/or gravel	3000.000	200.000																									
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Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	-0.003																										
Mz (kipft)	21.223	36.844																										
	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: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$$H_o = \frac{(-1.635 \text{ kip})}{1.57 \times (48 \text{ in})}$$H_o = -0.26035 \text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(21.223 \text{ kipft}) + ((-1.635 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.3795 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 5.2133 \text{ ft}$ - Required depth in x-direction, Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.2133 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.213 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.5 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.213 \text{ ft})}{(5.5 \text{ ft})}$ $Ratio = 0.94782$	Status: PASS Ratio: 0.950
	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 $q = \frac{P_v}{A}$ $q = \frac{(4.882 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.30513 \text{ kip/ft}^2$ Check bearing capacity ratio: <i>Ratio</i> - Capacity $Ratio = \frac{q}{q_o}$ $Ratio = \frac{(0.30513 \text{ kip/ft}^2)}{(3000 \text{ psf})}$ $Ratio = 0.10171$	Status: PASS Ratio: 0.100
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:

$H_o = -0.26035$ kip/ft - Lateral force per length of pile,

$M_o = 3.3795$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3795 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.26035 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (3.3795 \text{ kipft/ft})) + (4 \times (-0.26035 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7676 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.3795 \text{ kipft/ft})) + (3 \times (-0.26035 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^3 [(3 \times (3.3795 \text{ kipft/ft})) + (2 \times (-0.26035 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.28986 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3795 \text{ kipft/ft})) + ((-0.26035 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 1.0566 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(3.7676 \text{ ft})}{2}$$

$$p_a = 0.37676 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28986 \text{ kip/ft}^2)}{(0.37676 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76934$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 1.1 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

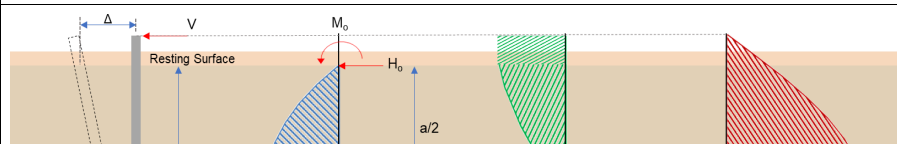
$$\text{Ratio} = \frac{s}{p_s}$$

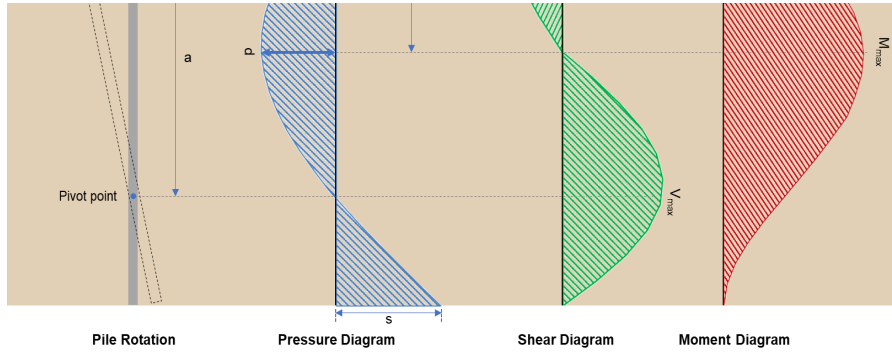
$$\text{Ratio} = \frac{(1.0566 \text{ kip/ft}^2)}{(1.1 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96054$$

Status: **PASS**
Ratio: **0.770**

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.725 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43392 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(36.844 \text{ kipft}) + ((-2.725 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.8669 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.8669 \text{ kipft/ft})}{(-0.43392 \text{ kip/ft})}$$

$$E = 13.521 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8669 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.43392 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (5.8669 \text{ kipft/ft})) + (4 \times (-0.43392 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7644 \text{ ft}$$

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

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

$$V_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.521 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7644 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.521 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7644 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.699 \text{ kip}$$

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

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

$$M_{max} = ((-0.43392 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(13.521 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7644 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.521 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7644 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.521 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7644 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 23.052 \text{ kipft}$	
		Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.003 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.00047771 \text{ kipft/ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.00047771 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.00047771 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.5 \text{ ft}))}$ $a = 3.6667 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$ $V_{max} = 12 \times \left(\frac{(0.00047771 \text{ kipft/ft}) \times (48 \text{ in})}{(5.5 \text{ ft})} \right) \times \left(\frac{(3.6667 \text{ ft})}{(5.5 \text{ ft})} - 1 \right) \times \left(\frac{(3.6667 \text{ ft})}{(5.5 \text{ ft})} \right)^2$ $V_{max} = 0.00061764 \text{ kip}$ M_{max} - Max bending moment at depth $a/2$, $M_{max} = (M_o b) \left[1 - \left(4 \frac{a}{2 L_e} \right)^3 \right] + \left(3 \frac{a}{2 L_e} \right)^4$ $M_{max} = ((0.00047771 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(3.6667 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left(3 \times \frac{(3.6667 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4$ $M_{max} = 0.0016985 \text{ kipft}$	
Table 22.4.2.1	22.4.2.2, 10.6.1.1	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, $A_{st,required}$ $A_{st,required} = Min \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = Min \left[\frac{\frac{(7.522 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.346 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = Max [A_{st,required}, (0.0018 A_g)]$ $A_{min} = Max [(-84.346 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ $n_{s,min}$ - Required number of reinforcement	

	<i>n_{rebar}</i> - Required number of reinforcement bars, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <i>A_{st}</i> - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <i>Ratio</i> - Capacity $Ratio = \frac{A_{min}}{A_{st}}$ $Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $Ratio = 0.96556$	Status: PASS Ratio: 0.970
25.2.3	<i>s_{rebar}</i> - Minimum spacing of reinforcement, $s_{rebar} = Max[1.5, (1.5 d_{bar})]$ $s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) 25.7.2.1 <i>s_{ties}</i> - Maximum spacing of ties, $s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$ $s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) <i>φP_N</i> - Allowable axial compressive strength $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <i>Ratio</i> - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.522 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0028118$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: <i>b_w</i> = 48 in - Effective width, <i>d</i> - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$	

		$d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor	$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.522 \text{ kip} \rightarrow 7522 \text{ lbf}$, $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7522 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.49 \text{ kip}$	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$	
	V_c - Governing shear strength of concrete	$V_c = MIN[V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN[(296.21 \text{ kip}), (119.49 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.49 \text{ kip}$	
22.5.1.2	$V_{s,a}$ - Shear strength of steel (a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$	
	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)	$V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$	
	V_s - Governing shear strength of steel	$V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$	

22.5.1.1	$V_s = 50.894 \text{ kip}$ $V_s = 50.894 \text{ kip}$ $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.49 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.75 \text{ kip}$ Considering x-direction: $V_{max} = 8.699 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(8.699 \text{ kip})}{(110.75 \text{ kip})}$ $Ratio = 0.078548$	Status: PASS Ratio: 0.080
14.5.2.1b	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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 23.052 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(23.052 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.092355$	Status: PASS Ratio: 0.090
	Considering z-direction: $M_{max} = 0.0016985 \text{ kipft}$ - Maximum moment in the z-direction, <i>Ratio</i> - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$	

	ψ_{max}	
	$Ratio = \frac{(0.0016985 \text{ kipft})}{(249.6 \text{ kipft})}$	
	$Ratio = 6.8049 \times 10^{-6}$	
		Status: PASS Ratio: 0.000