

## Project Details



**Project Name:** Solar Canopies - Montana - Example - **Date:** Mon Aug 19 2024

V1Jb

**Number of Modules:** 6

**Location:** Missoula, MT 59801, USA

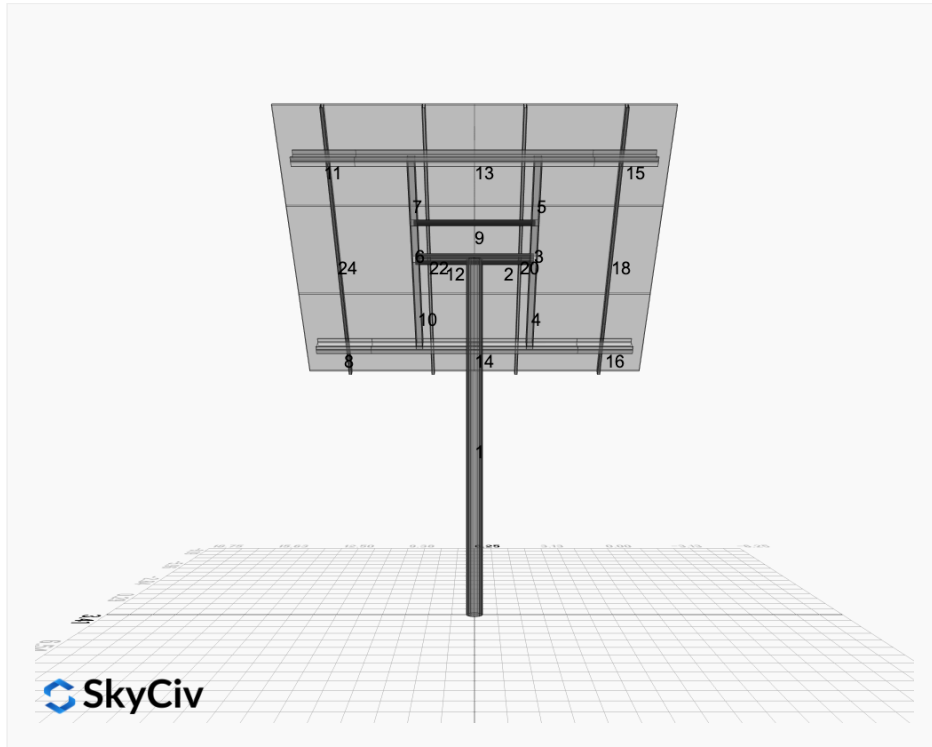
**Number of Poles:** 1

**Unique ID:** 1P-0-6TOP-SD-24-L-3Hx2W-127B

**Date Sold:**

**Dealer:** \_\_\_\_\_

\_\_\_\_\_



<b>Array Dimensions N/S</b>	10.63 ft
<b>Array Dimensions E/W</b>	12.50 ft
<b>Winter Tilt Angle</b>	50
<b>Front Edge Clearance</b>	8 ft

### MT Solar Bill of Materials (1P-0-6TOP-SD-24-L-3Hx2W-127B)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-24	24IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	2

### Rail Bill of Materials

Part	Qty
Rails (126in)	4
Rail Attachment	8
Module Mid Clamp	8
Module End Clamp	8
Ground Lug	2

## Site Details:



**Site Address:** Missoula, MT 59801, USA

### Array Specification

<b>Duty Classification:</b>	SD
<b>Module Width:</b>	42.00 in
<b>Module Length:</b>	74.00in
<b>Number of Rows:</b>	3
<b>Number of Columns:</b>	2
<b>Total Number of Modules:</b>	6
<b>Winter Tilt Angle:</b>	50
<b>Front Edge Clearance:</b>	8
<b>Total Array Height at Tilt:</b>	16.14 ft
<b>Total Frame Length:</b>	11.50 ft
<b>Frame Weight:</b>	707 lbs
<b>Array Dimensions N/S:</b>	10.63 ft
<b>Array Dimensions E/W:</b>	12.50 ft
<b>Rail Length:</b>	127.50 in
<b>Rail Spacing:</b>	3.13 ft

### Support Specifications

<b>Pole Size:</b>	6in Pipe Sch 40
<b>Pole Length above Grade:</b>	12.07 ft
<b>Number of Poles:</b>	1
<b>Pole Spacing:</b>	0

### Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 5.25 ft
<b>Foundation Volume:</b>	3.111 y <sup>3</sup>

### Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	Missoula, MT 59801, USA
<b>Wind Speed:</b>	99 mph
<b>Snow Load:</b>	34 psf

### **Design Disclaimer**

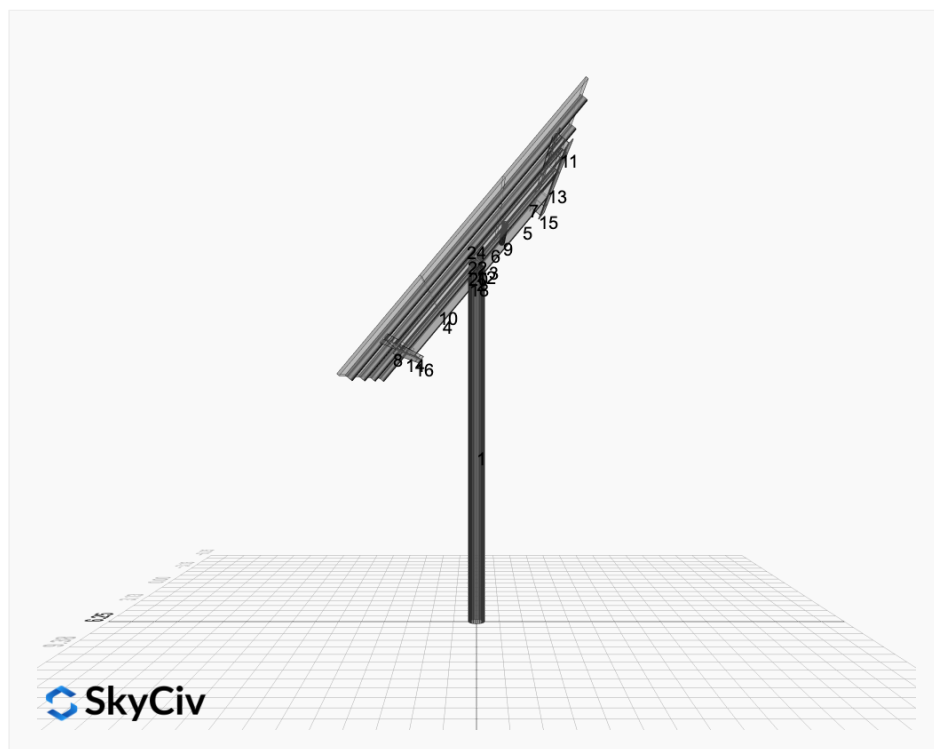
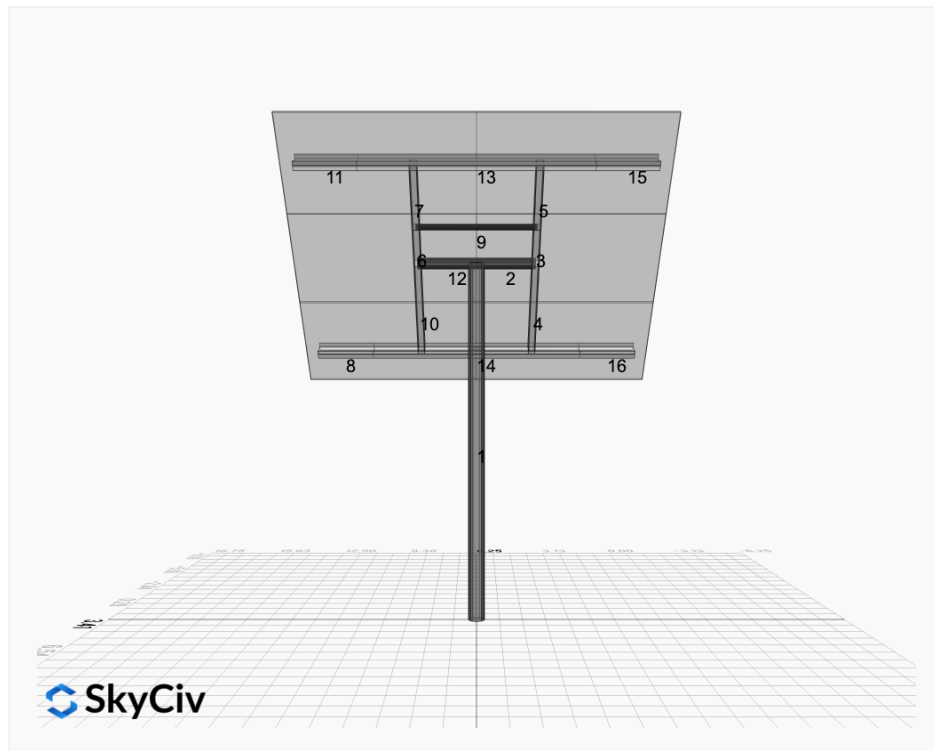
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

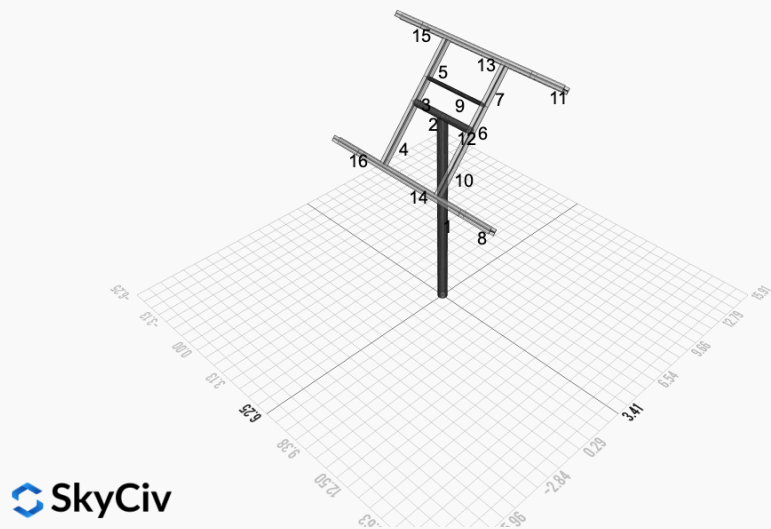
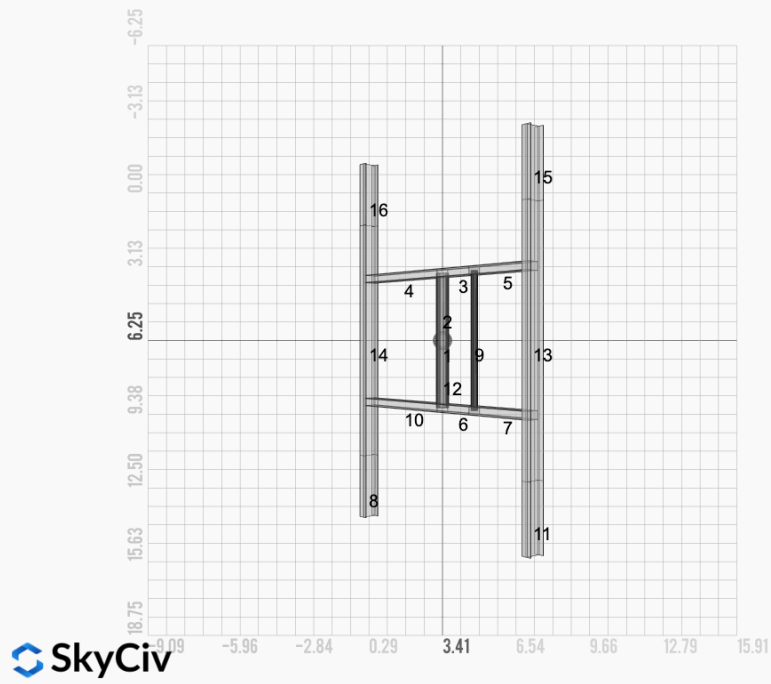
## AutoDesigner Input

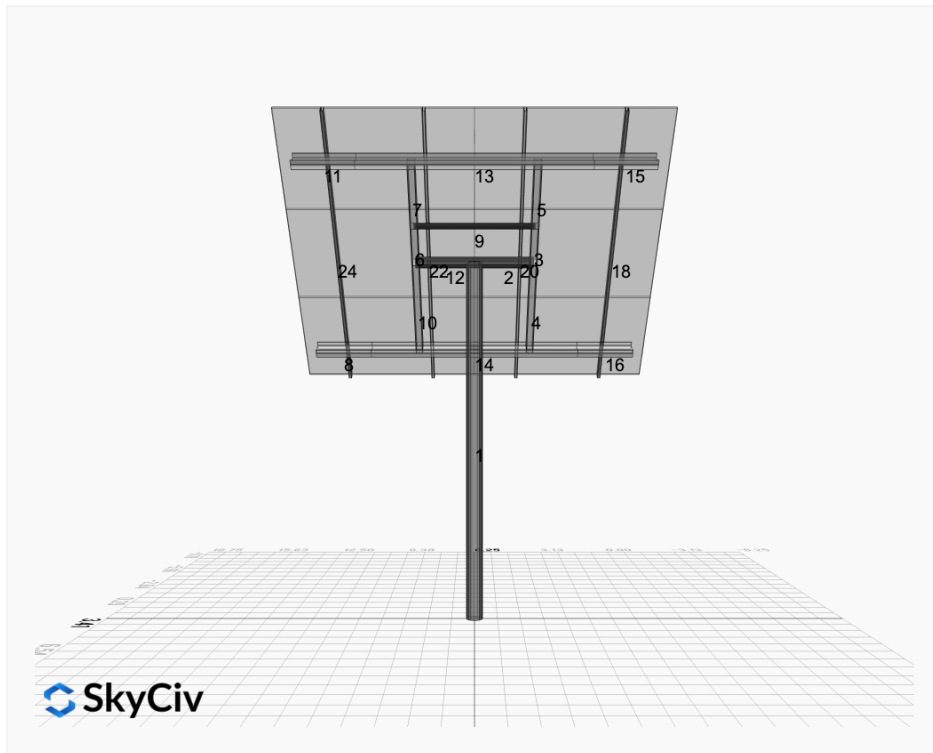
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### Design Notes:

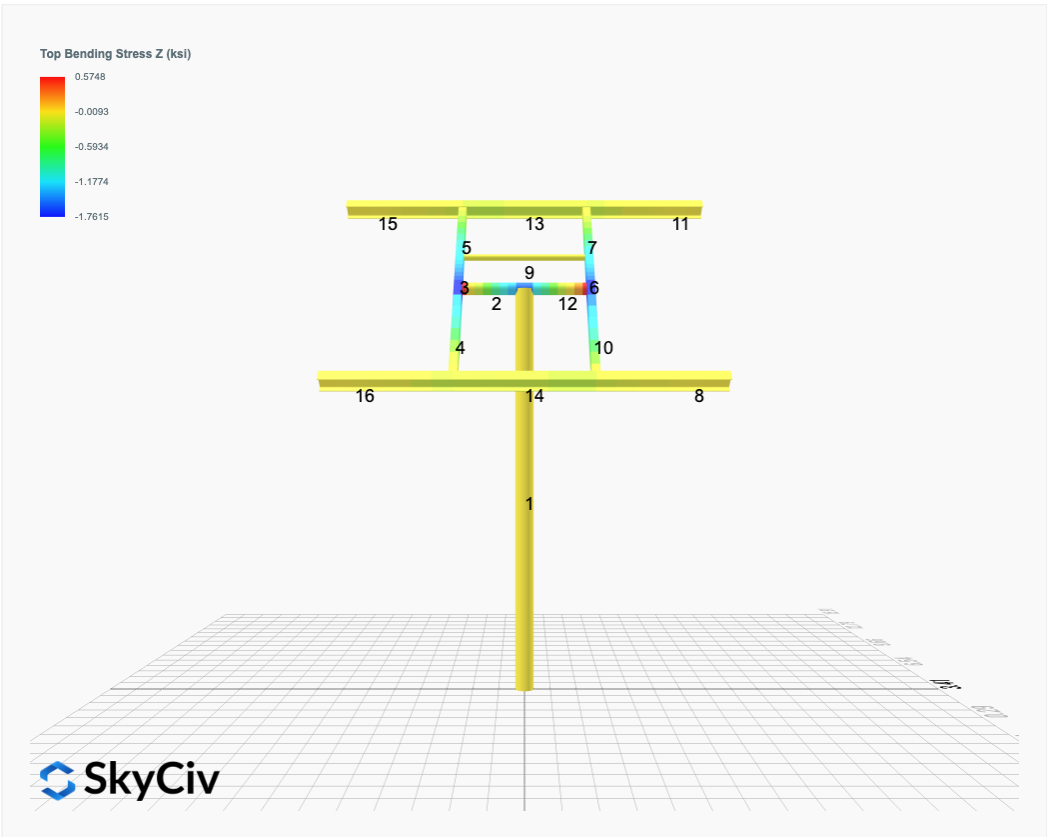
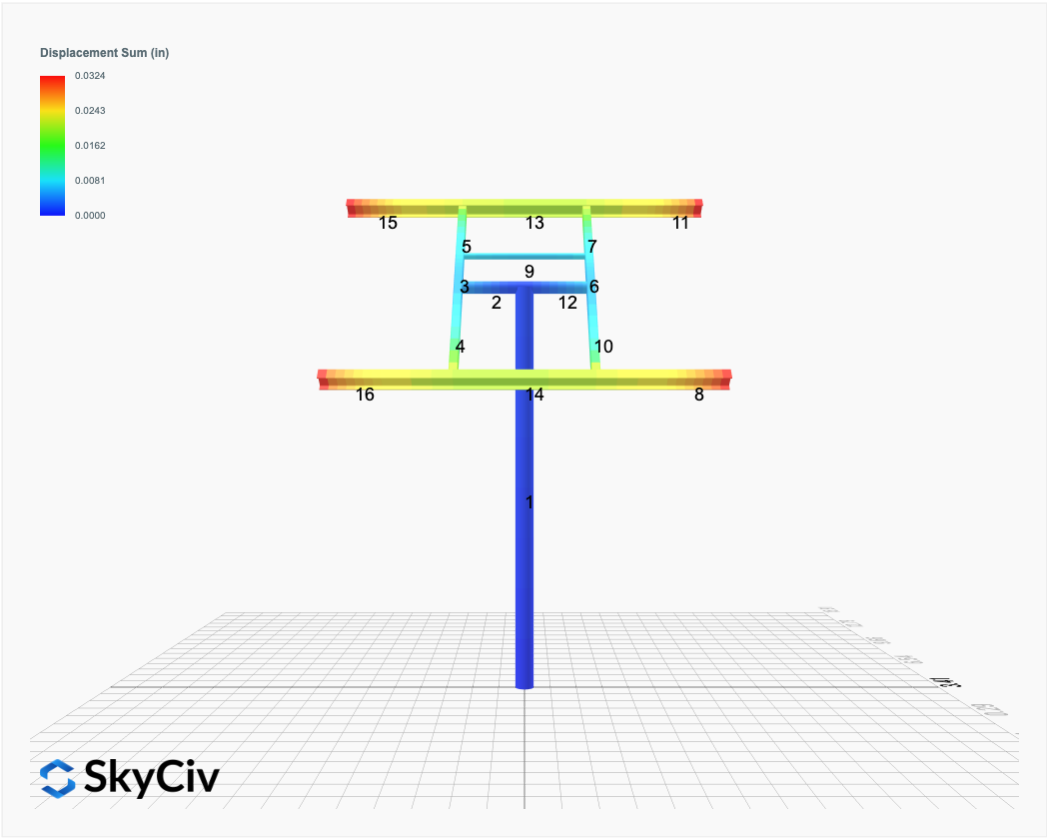
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

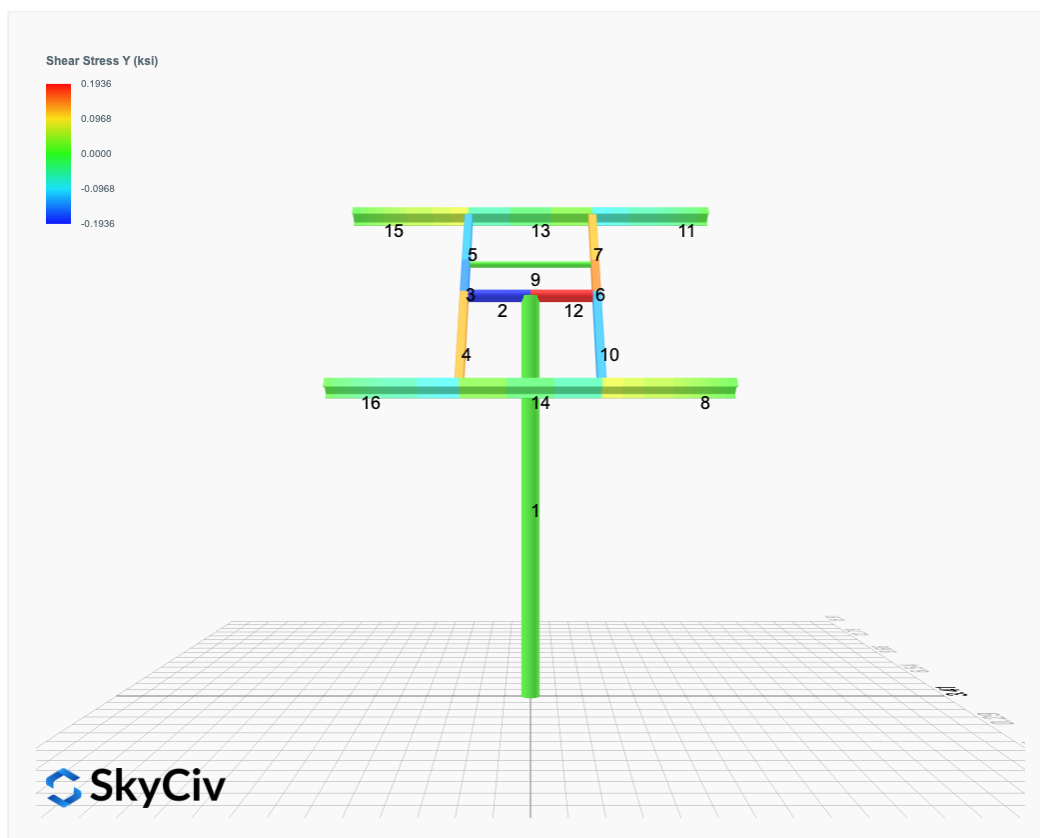
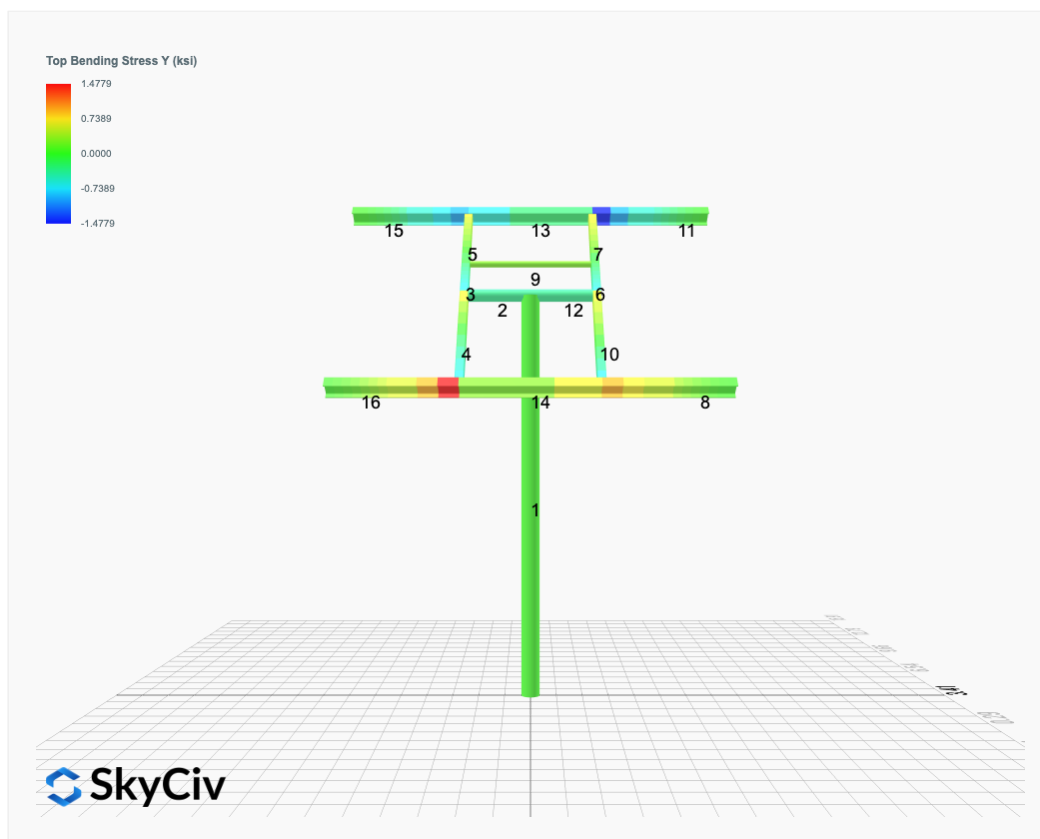


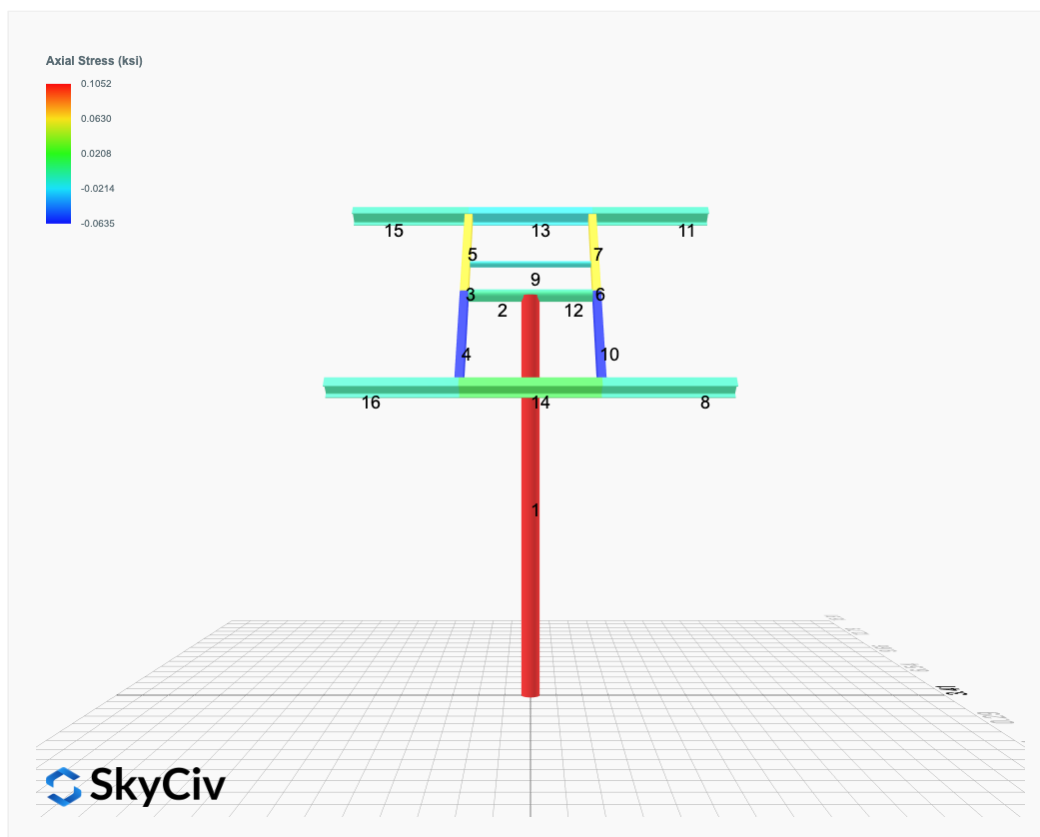




# FEM Results (Envelope Worst Case for each member)







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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 2. D + L	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 3. D + (S or Lr or R)	0.0000	1.8257	0.0000	0.0000	-0.0000	0.0149
ULS: 3. D + (S or Lr or R)	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 5b. D + 0.7E	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0148
ULS: 8. 0.6D + 0.7E	0.0000	0.7430	0.0000	0.0000	-0.0000	0.0086
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1170	2.1757	0.0000	0.0000	-0.0000	13.6800
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1170	0.3011	0.0000	0.0000	-0.0000	-13.2904
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8378	2.3818	0.0000	0.0000	-0.0000	10.2640
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8378	0.9759	0.0000	0.0000	-0.0000	-9.9637
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6789	0.0000	0.0000	-0.0000	0.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8378	1.9414	0.0000	0.0000	-0.0000	10.2636
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8378	0.5354	0.0000	0.0000	-0.0000	-9.9642
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.2384	0.0000	0.0000	-0.0000	0.0143
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1170	1.6803	0.0000	0.0000	-0.0000	13.6743
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.7430	0.0000	0.0000	-0.0000	0.0086
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1170	-0.1943	0.0000	0.0000	-0.0000	-13.2961
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.7430	0.0000	0.0000	-0.0000	0.0086

### 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	3.3419
Shear X	-1.8617
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	23.1673

### 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	2.3818
Shear X	-1.1170
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	13.6800

Project Details

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



Design Input Information

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

Design Materials			
ID	E (ksi)	$F_y$ (ksi)	$F_u$ (ksi)
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

Section Properties								
ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
-	-	-	-	-	-	-	-	-

1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties									
Member ID	Section ID	K <sub>z</sub> L (ft)	K <sub>y</sub> L (ft)	L <sub>b</sub> (ft)	C <sub>b</sub>	LS T	LS C	L D	
1	7	25.35	25.35	12.07	-	300	200	1	
2	4	1.30	1.30	2.00	-	300	200	1	
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.18,1.30,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1	
4	15	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.68,1.95,1.68,1.68,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1	
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.87,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.18,1.30,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	300	200	1	
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.87,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1	
8	18	4.20	4.20	2.00	2.33,2.33	300	200	1	
9	1	2.60	2.60	4.00	-	300	200	1	
10	15	2.44	2.44	3.75	1.69,1.69,1.69,1.68,1.69,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.68,1.95,1.68,1.68,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1	
11	18	4.20	4.20	2.00	2.33,2.33	300	200	1	
12	4	1.30	1.30	2.00	-	300	200	1	
13	18	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.09,1.09,1.09,1.09	300	200	1	
14	18	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.07,1.09,1.09,1.09,1.09,1.09,1.09,1.09	300	200	1	
15	18	4.20	4.20	2.00	2.33,2.33	300	200	1	
16	18	4.20	4.20	2.00	2.33,2.33	300	200	1	

## Member Design Capacity

Member ID	Φ <sub>t</sub> P <sub>n</sub> (kip)	Φ <sub>c</sub> P <sub>n</sub> (kip)	Φ <sub>b</sub> M <sub>zn</sub> (k-ft)	Φ <sub>b</sub> M <sub>yn</sub> (k-ft)	Φ <sub>v</sub> V <sub>yn</sub> (kip)	Φ <sub>v</sub> V <sub>zn</sub> (kip)
1	251.16	68.72	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	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	96.18	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	6.26	29.14	16.61
11	120.60	96.18	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.03	18.98	6.45	30.09	45.74
14	120.60	84.03	18.87	6.45	30.09	45.74

15	120.60	96.18	23.36	6.45	30.09	45.74
16	120.60	96.18	23.36	6.45	30.09	45.74

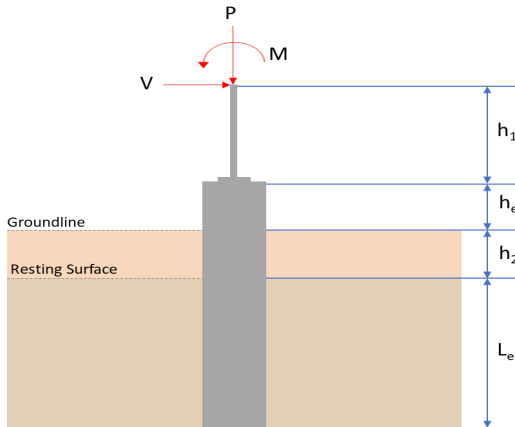
## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.049	0.548	0.000	0.025	0.000	0.572	#13	0.677	Not Required	Pass
2	0.000	0.154	0.114	0.036	0.022	0.268	#13	0.052	Not Required	Pass
3	0.005	0.286	0.052	0.029	0.013	0.324	#13	0.044	Not Required	Pass
4	0.005	0.285	0.047	0.029	0.009	0.317	#13	0.078	Not Required	Pass
5	0.005	0.177	0.046	0.029	0.008	0.182	#13	0.073	Not Required	Pass
6	0.005	0.286	0.052	0.029	0.013	0.324	#13	0.044	Not Required	Pass
7	0.005	0.177	0.046	0.029	0.008	0.182	#13	0.073	Not Required	Pass
8	0.000	0.012	0.020	0.009	0.003	0.030	#21	Not Required	Not Required	Pass
9	0.001	0.018	0.023	0.001	0.000	0.042	#13	0.132	Not Required	Pass
10	0.005	0.285	0.047	0.029	0.009	0.317	#13	0.078	Not Required	Pass
11	0.000	0.012	0.020	0.009	0.003	0.030	#21	Not Required	Not Required	Pass
12	0.000	0.154	0.114	0.036	0.022	0.268	#13	0.052	Not Required	Pass
13	0.001	0.059	0.072	0.018	0.005	0.111	#21	0.177	Not Required	Pass
14	0.002	0.061	0.072	0.018	0.005	0.111	#21	0.177	Not Required	Pass
15	0.000	0.012	0.020	0.009	0.003	0.030	#21	Not Required	Not Required	Pass
16	0.000	0.012	0.020	0.009	0.003	0.030	#21	Not Required	Not Required	Pass

## Definitions

Φ <sub>t</sub>	Safety factor for tensile
Φ <sub>c</sub>	Safety factor for compression
Φ <sub>b</sub>	Safety factor for flexure
Φ <sub>v</sub>	Safety factor for shear
E	Modulus of elasticity
F <sub>y</sub>	Specified minimum yield stress
F <sub>u</sub>	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I <sub>yp</sub>	Moment of inertia about the Y axes
I <sub>zp</sub>	Moment of inertia about the Z axes
I <sub>w</sub>	Warping constant
S <sub>yp</sub>	Plastic section modulus about the Y axis
S <sub>zp</sub>	Plastic section modulus about the Z axis
KL	Effective length
C <sub>b</sub>	Buckling modification factor (from all load combinations)
L <sub>b</sub>	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P <sub>n</sub>	Nominal axial strength (tension/compression)
M <sub>n</sub>	Nominal flexural strength (about Z/Y axis)
V <sub>n</sub>	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M <sub>z</sub>	Design ratio in case of bending about Z axis
M <sub>y</sub>	Design ratio in case of bending about Y axis
V <sub>y</sub>	Design ratio in case of shear along Y axis
V <sub>z</sub>	Design ratio in case of shear along Z axis
(P,M <sub>z</sub> ,M <sub>y</sub> )	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																										
	<div>SkyCiv Foundation Design</div> <div>Pile Foundation</div> <div>Design Information :</div> <div>Design code : IBC 2021 (International Building Code)</div> <div>Unit System : Imperial</div>																											
	<div>Pile Input</div> <div></div> <div>Geometry</div> <div>Pile shape: rectangular</div> <div>b = 48 in - Pile width</div> <div>D = 48 in - Pile depth</div> <div>L = 5.25 ft - Total pile length</div> <div>h1 = 0 ft - Lateral load height from the top of the pile,</div> <div>h2 = 0 ft - Depth to resisting surface</div> <div>he = 0 ft - Length of pile above the ground</div> <div>Tabulation of Soil Parameters</div> <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>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> <div>Tabulation of Loads</div> <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>2.382</td><td>3.342</td></tr><tr><td>Vx (kip)</td><td>-1.117</td><td>-1.862</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>0.000</td></tr><tr><td>Mz (kipft)</td><td>13.680</td><td>23.167</td></tr></table> <div>Material Properties</div> <div>f'ck = 2.5 ksi - Concrete strength,</div>	Layer	Label	Allowable Bearing Pressure (qa) (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)	2.382	3.342	Vx (kip)	-1.117	-1.862	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	13.680	23.167	
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	<div>Required depth to resist lateral loads (ASD)</div> <div>H - Point of application of the lateral load</div> <div><math display="block">H = h_1 + h_2 + h_e</math><math display="block">H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})</math><math display="block">H = 0 \text{ ft}</math></div> <div>Considering x-direction:</div> <div>Ho - Lateral force per length of pile,</div> <div><math display="block">H_o = \frac{V_x}{1.57 \text{ } D}</math><math display="block">H_o = \frac{(-1.117 \text{ kip})}{1.57 \times (48 \text{ in})}</math><math display="block">H_o = -0.17787 \text{ kip/ft}</math></div>																											

	<p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(13.68 \text{ kipft}) + ((-1.117 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.1783 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left( 14.14 \times \frac{H_o \times L_x}{R} \right) - \left( 18.85 \times \frac{M_o}{R} \right) = 0$ <p>Solving the cubic equation:  <math>L_{e,x} = 4.9518 \text{ ft}</math> - Required depth in x-direction,</p> <p><b>Considering z-direction:</b>  <math>L_{e,z} = 0 \text{ ft}</math> - Required depth in z-direction,</p> <p><b>Minimum embedded depth required:</b>  <math>L_{e,req}</math> - Depth of pile required,</p> $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(4.9518 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 4.952 \text{ ft}$ <p><math>L_e</math> - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.25 \text{ ft}$ <p>Ratio - Embedded depth</p> $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(4.952 \text{ ft})}{(5.25 \text{ ft})}$ $Ratio = 0.94324$	<p>Status: <b>PASS</b> Ratio: <b>0.940</b></p>
	<p><b>End-bearing Capacity (ASD)</b>  <math>A</math> - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p><math>q</math> - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(2.382 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.14888 \text{ kip/ft}^2$ <p><b>Check bearing capacity ratio:</b>  Ratio - Capacity</p> $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.14888 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.074438$	<p>Status: <b>PASS</b> Ratio: <b>0.070</b></p>
Czerniak	<p><b>Lateral Soil Pressure (ASD):</b>  <math>L/D</math> - Length to least lateral dimension ratio,</p>	

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since  $L/D \leq 10$ ,

Pile is short.

#### Considering x-direction:

$H_o = -0.17787 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 2.1783 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (2.1783 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.17787 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.1783 \text{ kipft/ft})) + (4 \times (-0.17787 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5972 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.1783 \text{ kipft/ft})) + (3 \times (-0.17787 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.1783 \text{ kipft/ft})) + (2 \times (-0.17787 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20377 \text{ kip/ft}^2$$

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

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

$$s = \frac{6 \times [(2 \times (2.1783 \text{ kipft/ft})) + ((-0.17787 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74512 \text{ kip/ft}^2$$

#### Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5972 \text{ ft})}{2}$$

$$p_a = 0.26979 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20377 \text{ kip/ft}^2)}{(0.26979 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75527$$

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

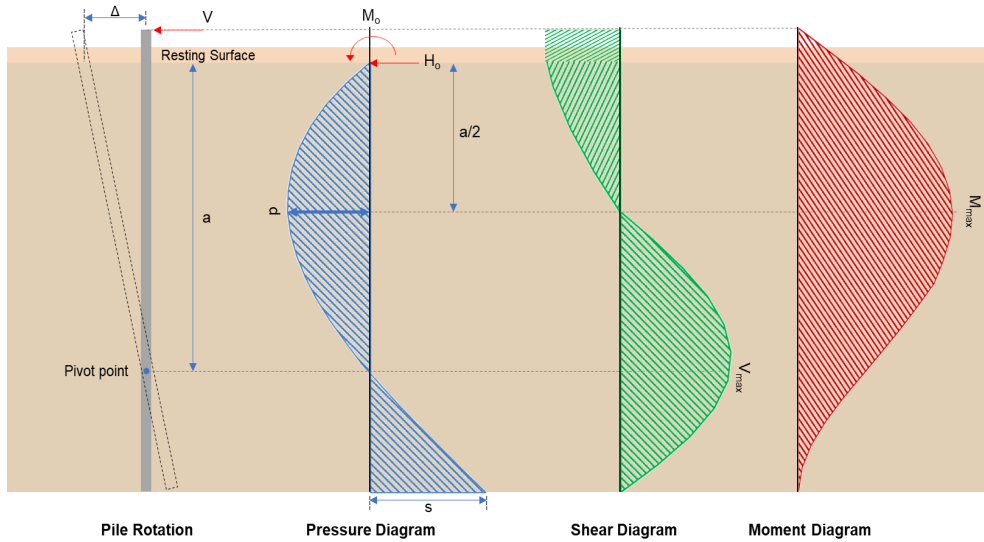
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.74512 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

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

$$Ratio = 0.94618$$

Status: **PASS**  
Ratio: **0.950**



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-1.862 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.2965 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(23.167 \text{ kipft}) + ((-1.862 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.689 \text{ kipft/ft}$$

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

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

$$E = \frac{(3.689 \text{ kipft/ft})}{(-0.2965 \text{ kip/ft})}$$

$$E = 12.442 \text{ ft}$$

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

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

$$a = \frac{(4 \times (3.689 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.2965 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.689 \text{ kipft/ft})) + (4 \times (-0.2965 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5961 \text{ ft}$$

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

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

$$V_{max} = ((-0.2965 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (12.442 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left( \frac{(3.5961 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (12.442 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left( \frac{(3.5961 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 3.1951 \text{ kip}$ <p><math>M_{max}</math> - Max bending moment located at depth <math>a/2</math>,</p> $M_{max} = (H_o D L_e) \left[ \left( \frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[ \left( \frac{4 E}{L_e} + 3 \right) \left( \frac{a}{2 L_e} \right)^3 + \left[ \left( \frac{3 E}{L_e} + 2 \right) \left( \frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.2965 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[ \left( \frac{(12.442 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5961 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[ \left( \frac{4 \times (12.442 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left( \frac{(3.5961 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (12.442 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left( \frac{(3.5961 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 14.547 \text{ kipft}$	
<p>Table 22.4.2.1</p> <p>22.4.2.2, 10.6.1.1</p>	<p><b>Minimum Reinforcement Check (LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>f'_{ck} = 2.5 \text{ ksi}</math> - Concrete strength,  <math>f_{yk} = 60 \text{ ksi}</math> - Longitudinal reinforcement strength,  <math>\phi = 0.65</math> - Reduction factor for axial strength,  <math>\alpha = 0.8</math> - Alpha factor for axial strength,  <math>A_g = 2304 \text{ in}^2</math> - Gross area of concrete,</p> <p><b>Longitudinal reinforcement:</b></p> <p>Required reinforcement due to axial load, <math>A_{st,required}</math></p> $A_{st,required} = \text{Min} \left[ \frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[ \frac{\frac{(3.342 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.485 \text{ in}^2$ <p><math>A_{min}</math> - Governing minimum reinforcement area,</p> $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.485 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ <p><math>n_{rebar}</math> - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <p><math>A_{st}</math> - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <p><b>Ratio</b> - Capacity</p> $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$	
25.2.3	<p><math>s_{rebar}</math> - Minimum spacing of reinforcement,</p> $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$	<p>Status: <b>PASS</b> Ratio: <b>0.970</b></p>

	$s_{rebar} = Max[1.5,(1.5 \times (0.625\text{ in}))]$ $s_{rebar} = 1.5\text{ in}$ <p><b>Ties:</b></p> <p>Since longitudinal reinforcement is <math>\leq</math> No. 10ø: Use #3(0.375 in)</p> <p><math>s_{ties}</math> - Maximum spacing of ties,</p> $s_{ties} = Min[(16 d_{bar}), (48 s_{ties}), Min(D,b)]$ $s_{ties} = Min[(16 \times (0.625\text{ in})), (48 \times (0.375\text{ in})), Min((48\text{ in}), (48\text{ in}))]$ $s_{ties} = 10\text{ in}$ <p><b>Summary:</b></p> <p>Main reinforcement: <b>14 - #5 (0.625 in)</b></p> <p>Ties: <b>#3(0.375 in) - 10 in</b></p>	
22.4.2.2	<p><b>Axial Compression Strength (ACI 318-19, LRFD)</b></p> <p><math>\phi P_N</math> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5\text{ ksi}) \times [(2304\text{ in}^2) - (4.2951\text{ in}^2)]) + ((60\text{ ksi}) \times (4.2951\text{ in}^2))]$ $\phi P_N = 2675.2\text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(3.342\text{ kip})}{(2675.2\text{ kip})}$ $Ratio = 0.0012493$	Status: PASS Ratio: 0.000
22.5.2.2	<p><b>Shear Strength (ACI 318-19, LRFD)</b></p> <p><b>Parameters:</b></p> <p><math>b_w = 48\text{ in}</math> - Effective width, <math>d</math> - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48\text{ in})$ $d = 38.4\text{ in}$	
22.5.5.1.3	<p><math>\lambda_s</math> - size effect modification factor</p> $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[ \sqrt{\frac{2}{1 + \frac{(38.4\text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}</math>,</p> <p><math>V_{c,max}</math> - 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)	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}</math>, <math>P = 3.342\text{ kip} \rightarrow 3342\text{ lbf}</math>,</p> <p><math>V_{c,a}</math> - Shear strength of concrete (a)</p> $V_{c,a} = \left[ 2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3342 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 118.93 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (118.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 118.93 \text{ kip}$$

22.5.1.2 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)

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

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

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

$V_s$  - Governing shear strength of steel

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

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

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

22.5.1.1  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((118.93 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.39 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 5.7581 \text{ kip}$  - Maximum shear force in the x-direction,  
 $Ratio$  - Capacity

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

$$Ratio = \frac{(5.7581 \text{ kip})}{(110.39 \text{ kip})}$$

$$Ratio = 0.052163$$

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

**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,

$\phi M_n$  - 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:**

$M_{max} = 14.547 \text{ kipft}$  - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.547 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.058283$$

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