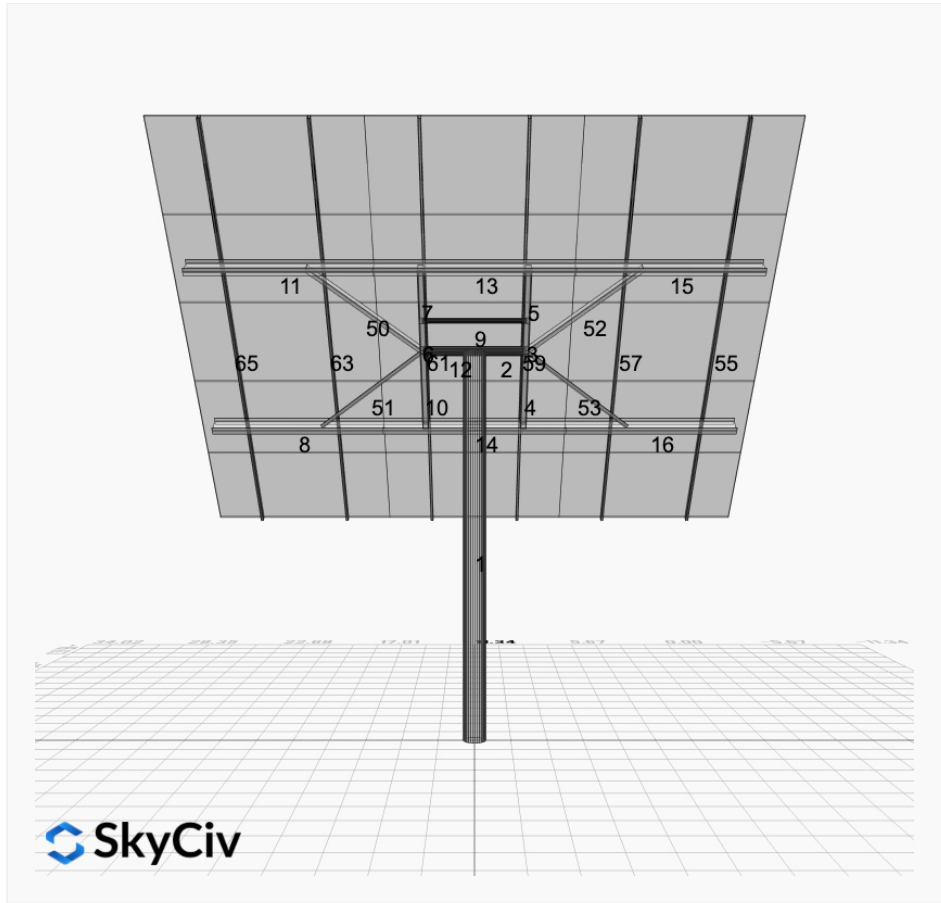


Project Name: SunUp Ranch - 5x3 - Round - V1Jb **Date:** Tue Dec 03 2024
Location: 9J3F+65, East Gull Lake, MN 56401, USA **Number of Modules:** 15
Unique ID: 1P-0-10TOP-HD-84-L-5Hx3W-STRUTS-G9EB **Number of Poles:** 1
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	22.68 ft
Winter Tilt Angle	50
Front Edge Clearance	8 ft

MT Solar Bill of Materials (1P-0-10TOP-HD-84-L-5Hx3W-STRUTS-G9EB)

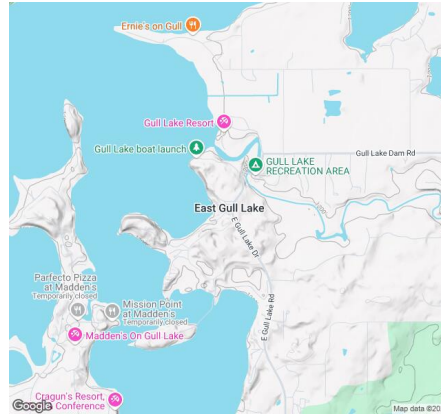
Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-84	84IN HD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	3

Rail Bill of Materials

Part	Qty
Rails (223in)	6
Rail Attachment	24
Module Mid Clamp	24
Module End Clamp	12

Part	Qty
Ground Lug	3

Site Details:



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

Array Specification

Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Winter Tilt Angle:	50
Front Edge Clearance:	8
Total Array Height at Tilt:	22.41 ft
Total Frame Length:	21.50 ft
Frame Weight:	1650 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	22.68 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	15.21 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 15.00 ft
Foundation Volume:	3.927 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

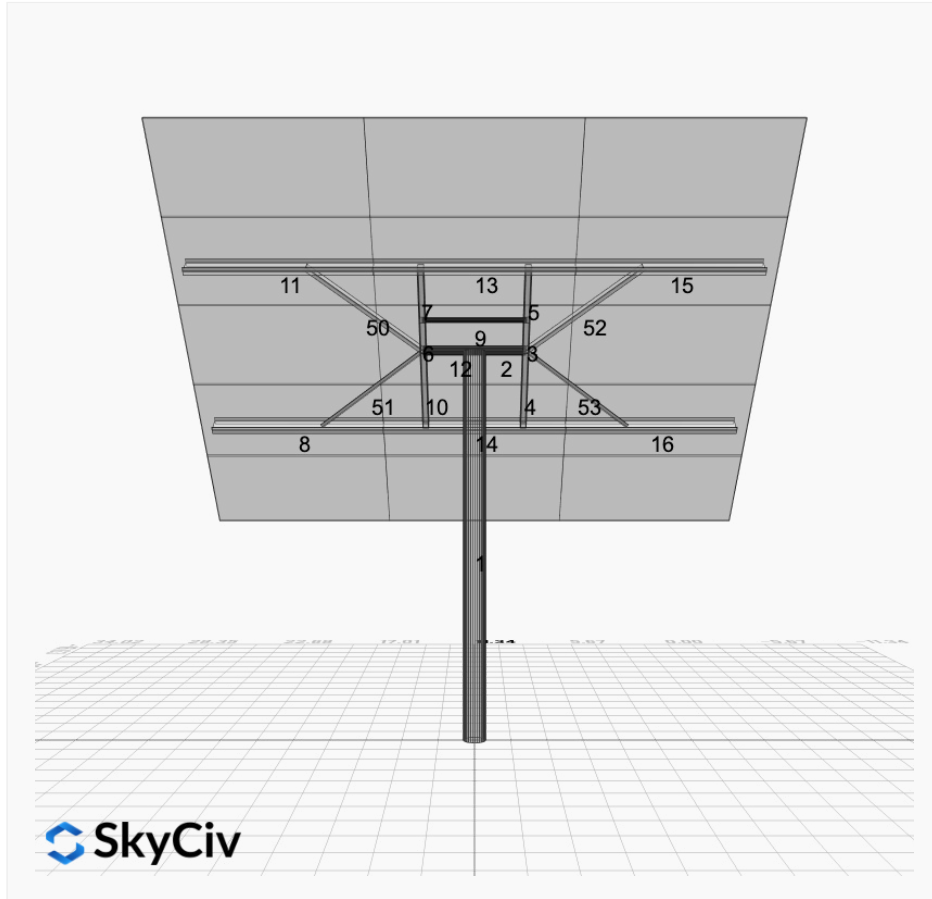
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

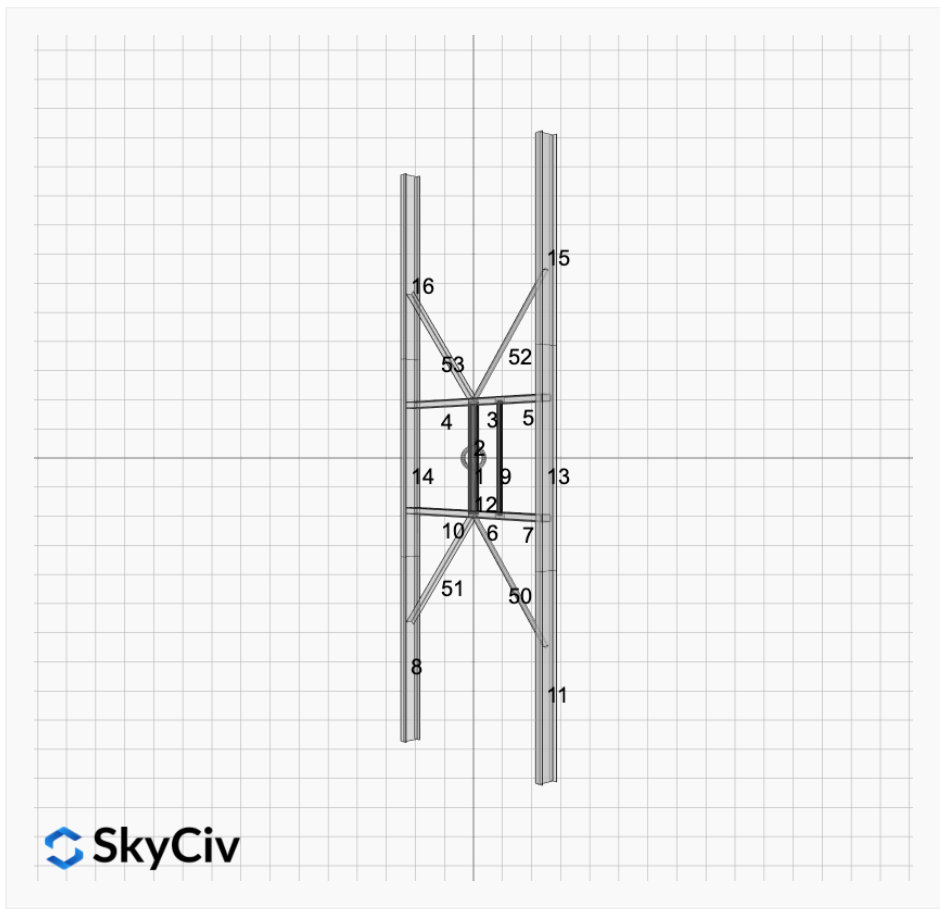
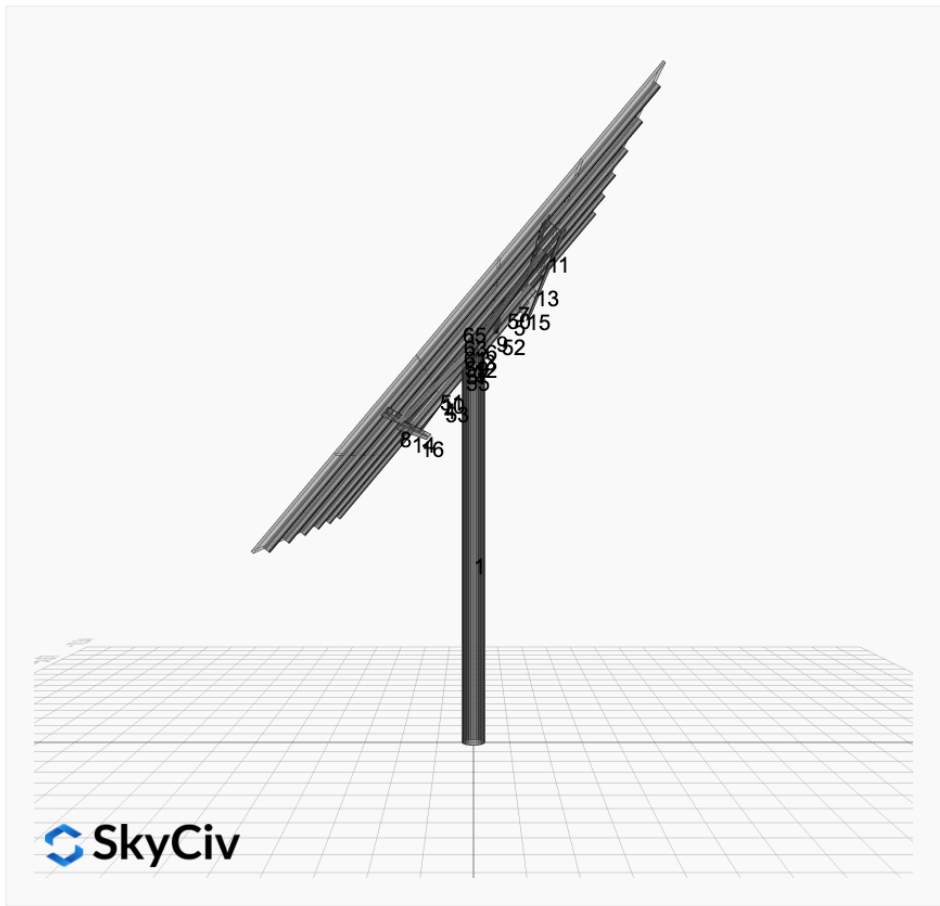
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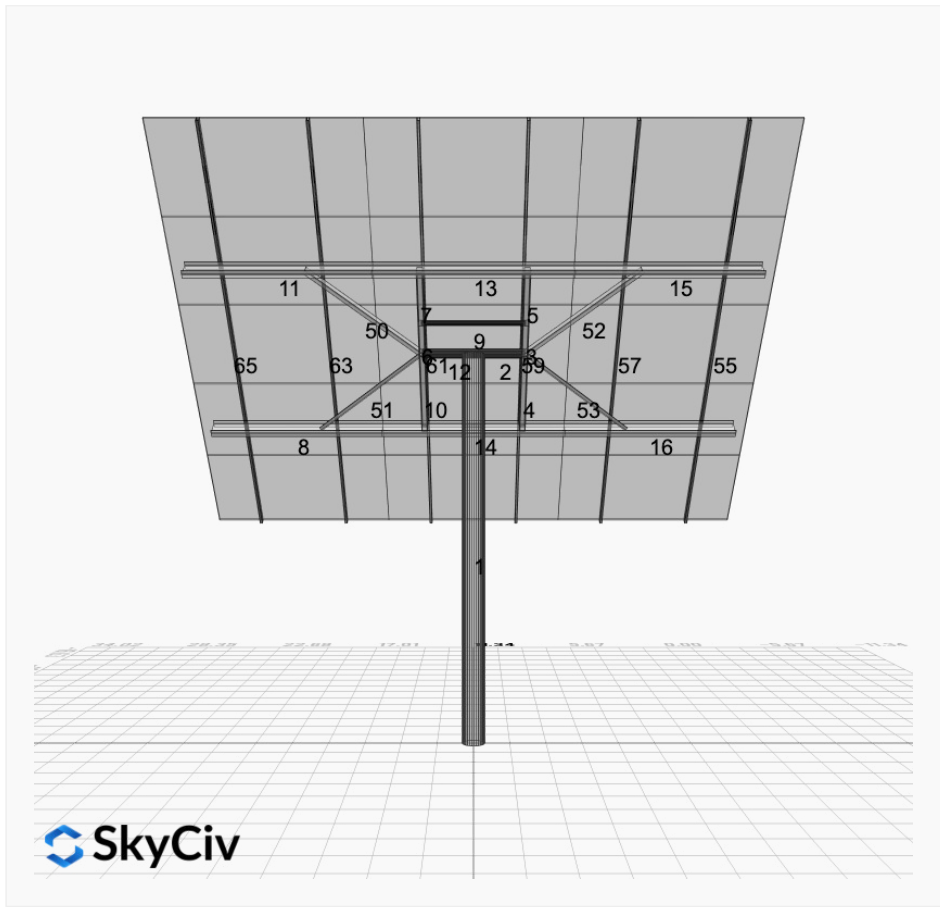
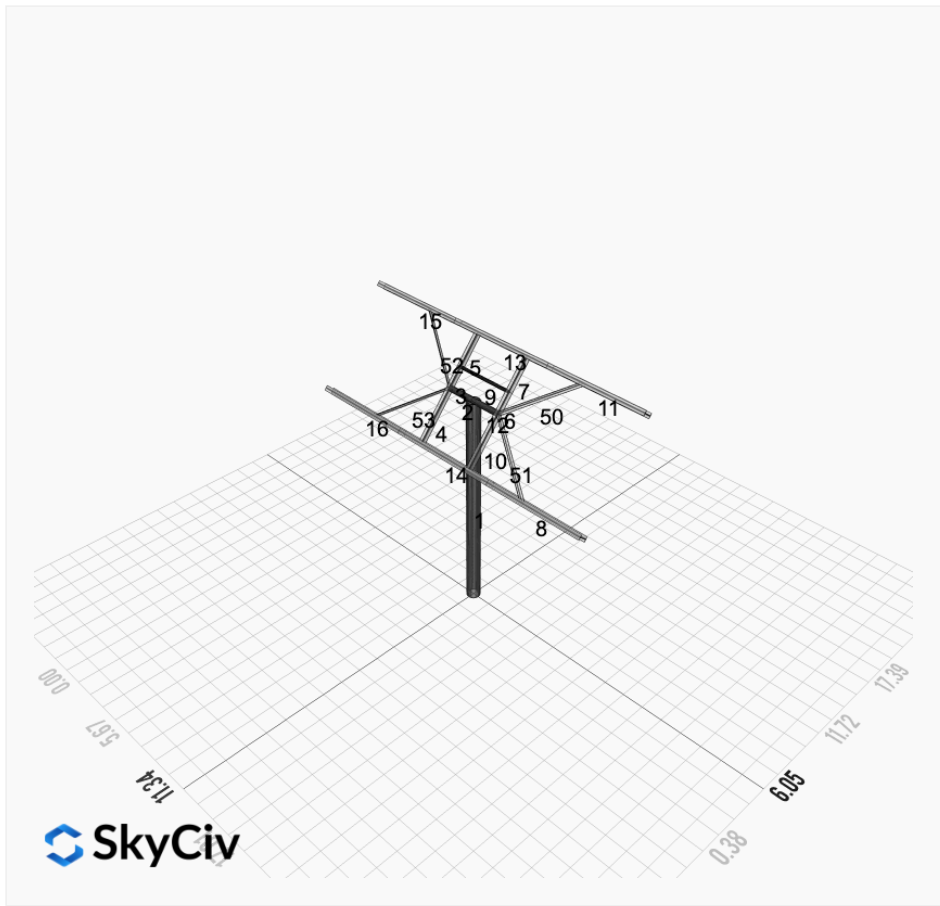
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Design Notes:

- 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
- 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)

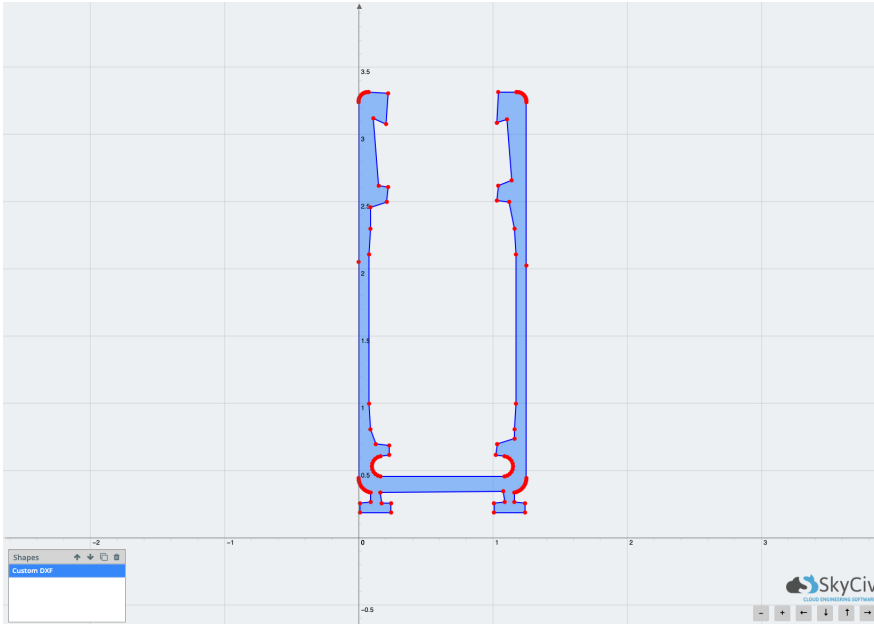






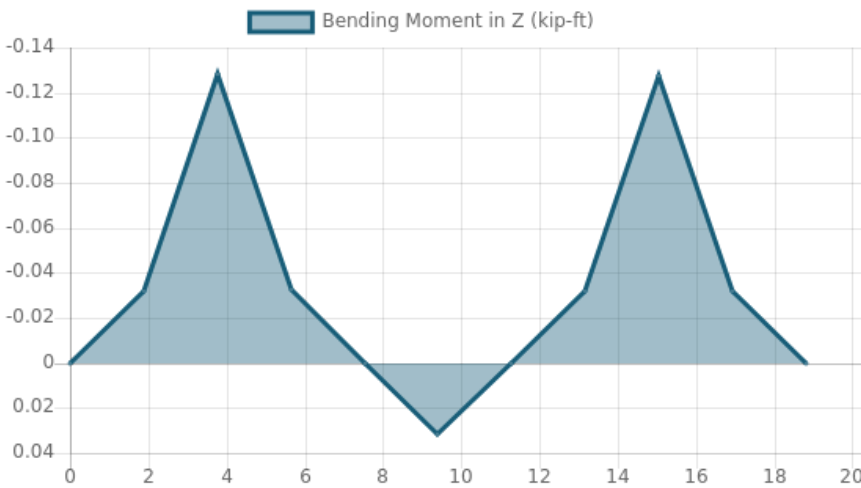
Rail Design Check

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): 0.0321 kip/ft
Snow (Y): -0.0382 kip/ft
Wind uplift Case A: 0.0563 kip/ft
Wind downforce Case A: 0.0563 kip/ft
Dead (Panel load) (X): 0.0107 kip/ft
Dead (Panel load) (Y): -0.0128 kip/ft

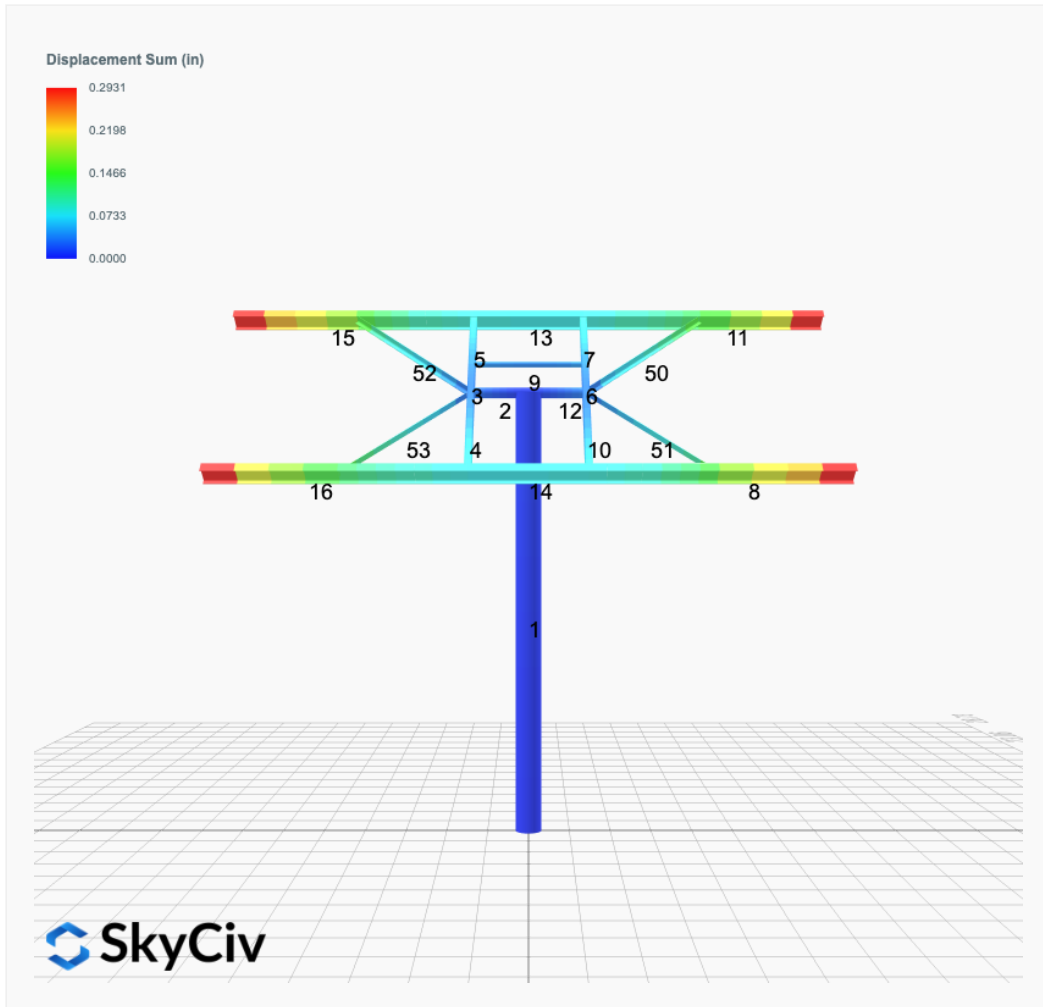


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	16.28170035	0.472	PASS
Material Yield	34.5	16.28170035	0.472	PASS
Material Strength	37	16.28170035	0.440	PASS

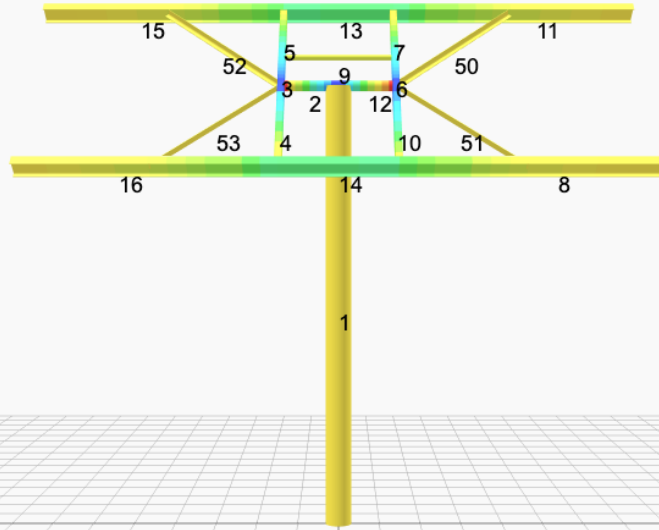
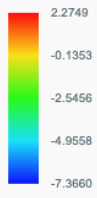
Member 1, ULS: 1. 1.4D



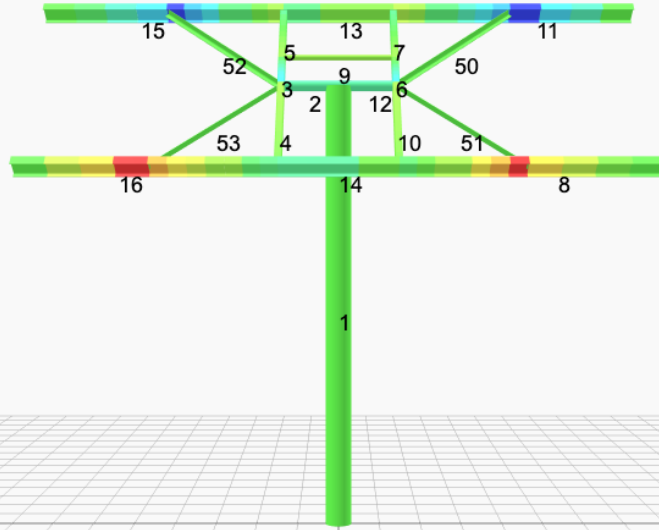
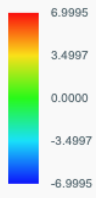
FEM Results (Envelope Worst Case for each member)



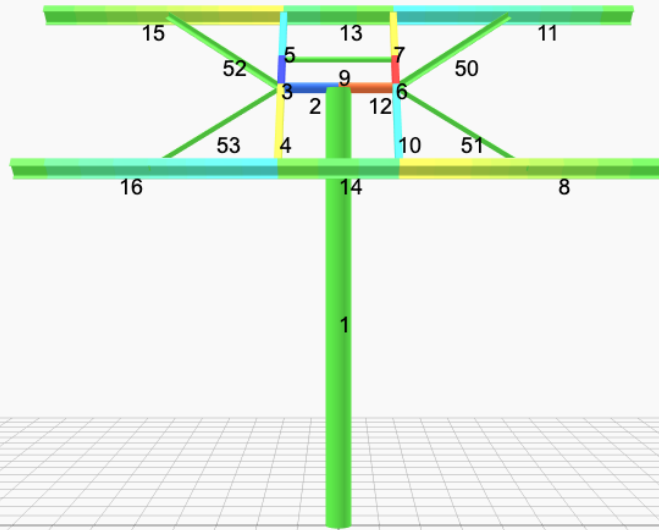
Top Bending Stress Z (ksi)



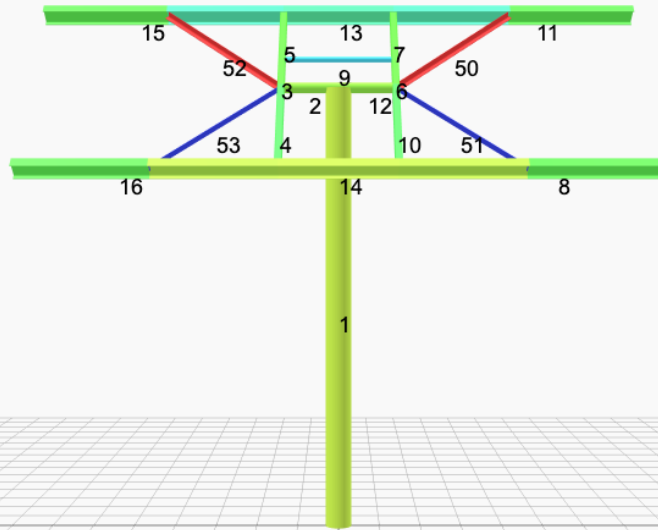
Top Bending Stress Y (ksi)



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	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 2. D + L	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 3. D + (S or Lr or R)	0.0000	6.6492	0.0000	-0.0000	-0.0000	0.0527
ULS: 3. D + (S or Lr or R)	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	5.7915	0.0000	-0.0000	-0.0000	0.0462
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 5b. D + 0.7E	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	5.7915	0.0000	-0.0000	-0.0000	0.0462
ULS: 8. 0.6D + 0.7E	0.0000	1.9311	0.0000	-0.0000	-0.0000	0.0160
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.9207	5.6692	0.0000	-0.0000	-0.0000	44.8746
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9207	0.7677	0.0000	-0.0000	-0.0000	-43.9572
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1905	7.6295	0.0000	-0.0000	-0.0000	33.6821
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	5.7915	0.0000	-0.0000	-0.0000	0.0462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1905	3.9534	0.0000	-0.0000	-0.0000	-32.9418
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	5.7915	0.0000	-0.0000	-0.0000	0.0462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1905	5.0565	0.0000	-0.0000	-0.0000	33.6627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1905	1.3804	0.0000	-0.0000	-0.0000	-32.9612
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.2185	0.0000	-0.0000	-0.0000	0.0267
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.9207	4.3818	0.0000	-0.0000	-0.0000	44.8639
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.9311	0.0000	-0.0000	-0.0000	0.0160
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9207	-0.5197	0.0000	-0.0000	-0.0000	-43.9679
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.9311	0.0000	-0.0000	-0.0000	0.0160

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	11.3936
Shear X	-4.8678
Shear Z	0.0000
Moment X	-0.0012
Moment Y (Twist)	0.0006
Moment Z	75.9430

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	7.6295
Shear X	-2.9207
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	44.8746

Project Details

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

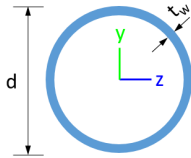


Design Input Information

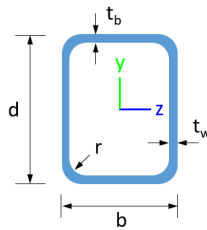
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

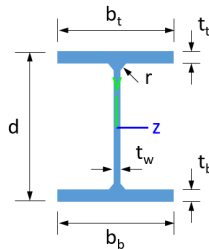
Section Dimensions



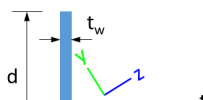
ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
11	10in Pipe Sch 40	10.75	0.36				



ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30



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	535.87	241.90	147.68	147.68	160.76	160.76
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	64.15	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	64.15	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	23.29	6.12	40.24	43.62
14	133.20	85.85	23.29	6.12	40.24	43.62
15	133.20	64.15	32.87	6.12	40.24	43.62
16	133.20	64.15	32.87	6.12	40.24	43.62
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

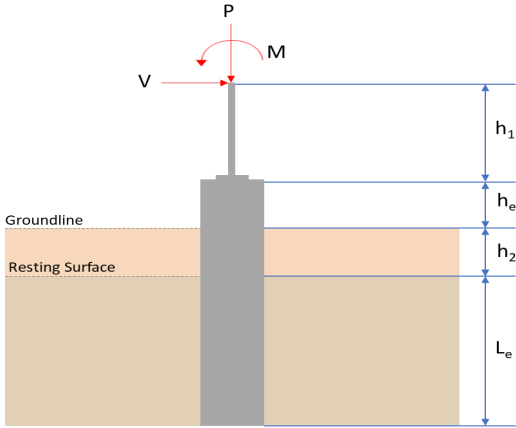
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.047	0.514	0.000	0.030	0.000	0.534	#13	0.521	Not Required	Pass
2	0.007	0.385	0.234	0.090	0.041	0.576	#13	0.053	Not Required	Pass
3	0.002	0.560	0.158	0.056	0.066	0.676	#21	0.045	Not Required	Pass
4	0.002	0.555	0.048	0.056	0.004	0.583	#13	0.080	Not Required	Pass
5	0.002	0.348	0.038	0.056	0.015	0.368	#13	0.074	Not Required	Pass
6	0.002	0.561	0.158	0.056	0.066	0.676	#21	0.045	Not Required	Pass
7	0.002	0.348	0.038	0.056	0.015	0.368	#13	0.074	Not Required	Pass
8	0.024	0.162	0.244	0.038	0.016	0.317	#21	0.500	Not Required	Pass
9	0.028	0.048	0.058	0.001	0.000	0.114	#13	0.136	Not Required	Pass
10	0.002	0.555	0.048	0.056	0.004	0.583	#13	0.080	Not Required	Pass
11	0.012	0.162	0.244	0.038	0.017	0.311	#21	0.333	Not Required	Pass
12	0.007	0.385	0.234	0.090	0.041	0.576	#13	0.053	Not Required	Pass
13	0.012	0.365	0.046	0.047	0.008	0.386	#13	0.190	Not Required	Pass
14	0.019	0.368	0.036	0.047	0.008	0.377	#13	0.286	Not Required	Pass
15	0.012	0.162	0.244	0.038	0.017	0.311	#21	0.333	Not Required	Pass
16	0.024	0.162	0.244	0.038	0.016	0.317	#21	0.333	Not Required	Pass
50	0.251	0.009	0.005	0.002	0.001	0.263	#21	0.783	Not Required	Pass
51	0.050	0.004	0.014	0.001	0.001	0.064	#21	0.522	Not Required	Pass
52	0.251	0.009	0.005	0.002	0.001	0.263	#23	0.783	Not Required	Pass
53	0.050	0.004	0.014	0.001	0.001	0.064	#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																										
	<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: round $D = 36$ in - Pile diameter $L = 15$ 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 1061 1227 1162"> <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 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.630</td> <td>11.394</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.921</td> <td>-4.868</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>-0.001</td> </tr> <tr> <td>M_z (kipft)</td> <td>44.875</td> <td>75.943</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.630	11.394	V_x (kip)	-2.921	-4.868	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	-0.001	M_z (kipft)	44.875	75.943	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
Load Component	ASD	LRFD																										
P (kip)	7.630	11.394																										
V_x (kip)	-2.921	-4.868																										
V_z (kip)	0.000	0.000																										
M_x (kipft)	0.000	-0.001																										
M_z (kipft)	44.875	75.943																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.921 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.97367 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(44.875 \text{ kipft}) + ((-2.921 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.958 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.9021 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(9.9021 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 9.902 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (15 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 15 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(9.902 \text{ ft})}{(15 \text{ ft})}$$

$$\text{Ratio} = 0.66013$$

Status: **PASS**
Ratio: **0.660**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.63 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.0794 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.0794 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.53971$$

Status: **PASS**
Ratio: **0.540**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(15 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.97367$ kip/ft - Lateral force per length of pile,

$M_o = 14.958$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.958 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-0.97367 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (14.958 \text{ kipft/ft})) + (4 \times (-0.97367 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.493 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_c)]^2}{L_c^2 [(3 M_o) + (2 H_o L_c)]}$$

$$p = \frac{1.178 \times [(4 \times (14.958 \text{ kipft/ft})) + (3 \times (-0.97367 \text{ kip/ft}) \times (15 \text{ ft}))]^2}{(15 \text{ ft})^2 \times [(3 \times (14.958 \text{ kipft/ft})) + (2 \times (-0.97367 \text{ kip/ft}) \times (15 \text{ ft}))]}$$

$$p = 0.085756 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_c ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_c)]}{L_c^2}$$

$$s = \frac{9.425 \times [(2 \times (14.958 \text{ kipft/ft})) + ((-0.97367 \text{ kip/ft}) \times (15 \text{ ft}))]}{(15 \text{ ft})^2}$$

$$s = 0.64139 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(10.493 \text{ ft})}{2}$$

$$p_a = 0.78696 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.085756 \text{ kip/ft}^2)}{(0.78696 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.10897$$

p_s - Allowable lateral soil pressure at depth L_c ,

$$p_s = R L_c$$

$$p_s = (150 \text{ psf/ft}) \times (15 \text{ ft})$$

$$p_s = 2.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

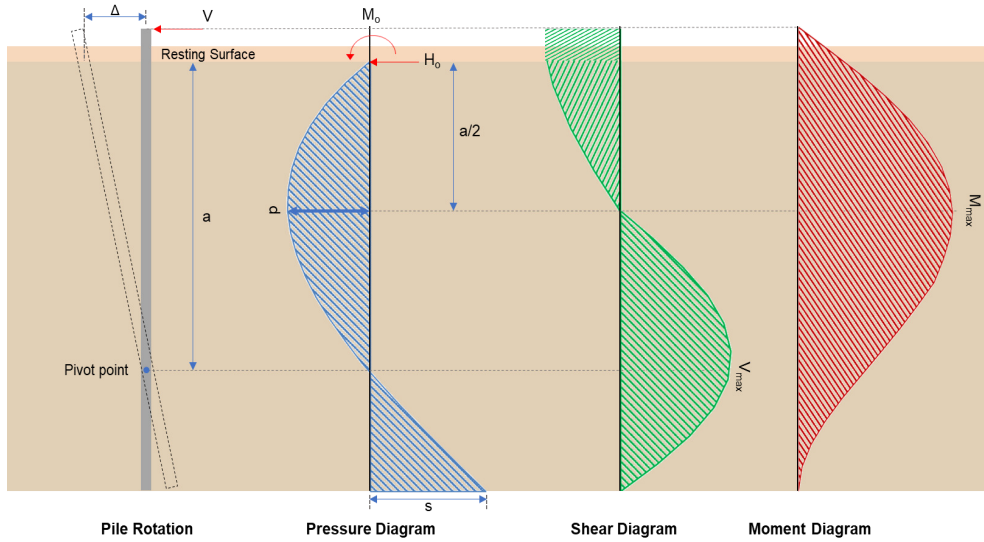
$$\dots (0.64139 \text{ kip/ft}^2)$$

Status: **PASS**
Ratio: **0.110**

$$\text{Ratio} = \frac{\dots}{(2.25 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.28506$$

Status: **PASS**
Ratio: **0.290**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.868 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.6227 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(75.943 \text{ kipft}) + ((-4.868 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 25.314 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(25.314 \text{ kipft/ft})}{(-1.6227 \text{ kip/ft})}$$

$$E = 15.6 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (25.314 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (-1.6227 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (25.314 \text{ kipft/ft})) + (4 \times (-1.6227 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.488 \text{ ft}$$

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

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

$$V_{max} = ((-1.6227 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.6 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(10.488 \text{ ft})}{(15 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (15.6 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(10.488 \text{ ft})}{(15 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.173 \text{ kip}$$

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

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

$$M_{max} = ((-1.6227 \text{ kip/ft}) \times (36 \text{ in}) \times (15 \text{ ft})) \times \left[\left(\frac{(15.6 \text{ ft})}{(15 \text{ ft})} + \frac{(10.488 \text{ ft})}{2 \times (15 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.6 \text{ ft})}{(15 \text{ ft})} + 3 \right) \times \left(\frac{(10.488 \text{ ft})}{2 \times (15 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (15.6 \text{ ft})}{(15 \text{ ft})} + 2 \right) \times \left(\frac{(10.488 \text{ ft})}{2 \times (15 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 84.715 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.00033333 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00033333 \text{ kipft/ft}) \times (15 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (15 \text{ ft})^2)}{(6 \times (0.00033333 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00033333 \text{ kipft/ft}) \times (36 \text{ in})}{(15 \text{ ft})} \right) \times \left(\frac{(10 \text{ ft})}{(15 \text{ ft})} - 1 \right) \times \left(\frac{(10 \text{ ft})}{(15 \text{ ft})} \right)^2$$

$$V_{max} = 0.00011852 \text{ kip}$$

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

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

$$M_{max} = ((0.00033333 \text{ kipft/ft}) \times (36 \text{ in})) \times \left[1 - \left(4 \times \frac{(10 \text{ ft})}{2 \times (15 \text{ ft})} \right)^3 + \left(3 \times \frac{(10 \text{ ft})}{2 \times (15 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 0.00088889 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\frac{(11.394 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.017 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-37.017 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$Ratio = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

25.2.3 s_{rebar} - 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. 10 \emptyset : Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**

Ties: **#3(0.375 in) - 10 in**

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$$

$$\phi P_N = 1253.9 \text{ kip}$$

Ratio - Capacity

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

$$Ratio = \frac{(11.394 \text{ kip})}{(1253.9 \text{ kip})}$$

$$Ratio = 0.0090868$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 $b_w = 36$ in - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

22.5.5.1.1 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5$ ksi \rightarrow 2500 psi,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5$ ksi \rightarrow 2500 psi, $P = 11.394$ kip \rightarrow 11394 lbf,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(11394 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.372 \text{ kip}$$

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

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

$$V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = MIN [(186.09 \text{ kip}), (76.372 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.372 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.372 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.453 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.173 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.173 \text{ kip})}{(74.453 \text{ kip})}$$

$$\text{Ratio} = 0.1635$$

Status: **PASS**
Ratio: **0.160**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,
 ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 84.715 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(84.715 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 1.3658$$

Status: **FAIL**
Ratio: **1.370**

Considering z-direction:

$M_{max} = 0.00088889 \text{ kipft}$ - Maximum moment in the z-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.00088889 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.000014331$$

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