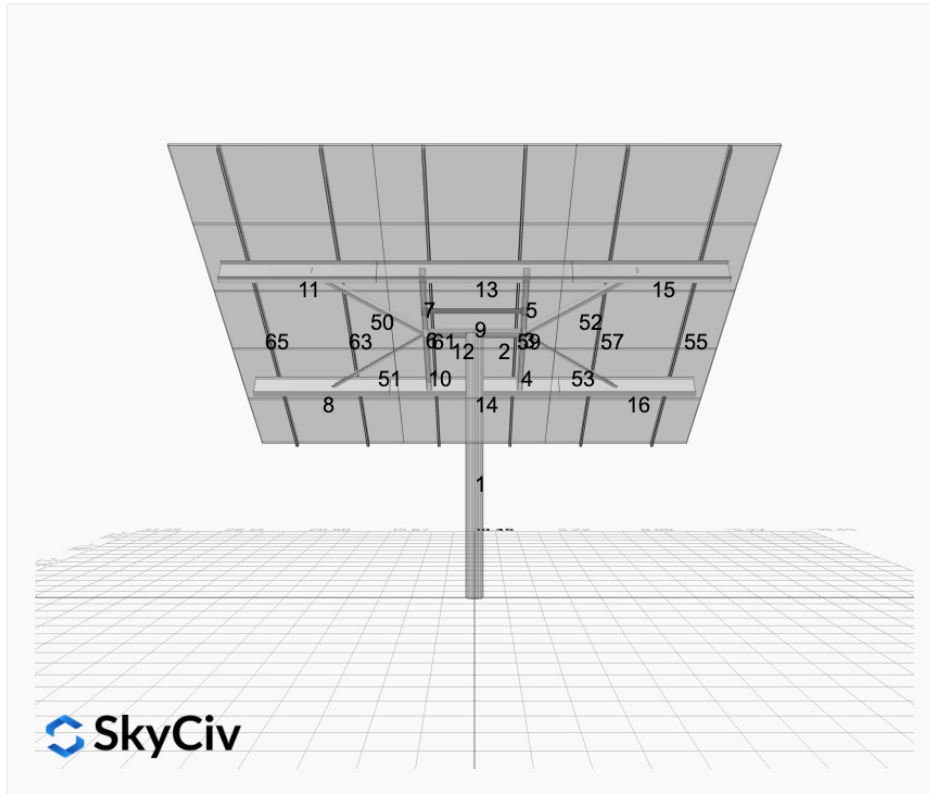


**Project Name:** Tiskus - Easthampton DE RE 35deg Struts - V1Jb  
**Location:** 94 West St, Easthampton, MA 01027, USA  
**Unique ID:** 1P-0-8TOP-XD-72-L-5Hx3W-STRUTS-F93A  
**Dealer:** \_\_\_\_\_

**Date:** Wed May 14 2025  
**Number of Modules:** 15  
**Number of Poles:** 1  
**Date Sold:** \_\_\_\_\_



<b>Array Dimensions N/S</b>	18.80 ft
<b>Array Dimensions E/W</b>	20.90 ft
<b>Winter Tilt Angle</b>	35
<b>Front Edge Clearance</b>	5.5 ft

### MT Solar Bill of Materials (1P-0-8TOP-XD-72-L-5Hx3W-STRUTS-F93A)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-XD	H-Frame Assembly-XD	1
MTS-XD-Wing-72	72IN XD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	3

### Rail Bill of Materials

Part	Qty
Rails (226in)	6
Rail Attachment	24
Module Mid Clamp	24
Module End Clamp	12
Ground Lug	3

# Site Details:



**Site Address:** 94 West St, Easthampton, MA 01027, USA

## Array Specification

<b>Duty Classification:</b>	XD
<b>Module Width:</b>	44.61 in
<b>Module Length:</b>	82.60in
<b>Number of Rows:</b>	5
<b>Number of Columns:</b>	3
<b>Total Number of Modules:</b>	15
<b>Winter Tilt Angle:</b>	35
<b>Front Edge Clearance:</b>	5.5
<b>Total Array Height at Tilt:</b>	16.28 ft
<b>Total Frame Length:</b>	19.50 ft
<b>Module Info/Notes:</b>	SILFAB SIL-530 XM
<b>Array Dimensions N/S:</b>	18.80 ft
<b>Array Dimensions E/W:</b>	20.90 ft
<b>Rail Length:</b>	225.55 in
<b>Rail Spacing:</b>	3.48 ft

## Support Specifications

<b>Pole Size:</b>	8in Pipe Sch 40
<b>Pole Length above Grade:</b>	10.89 ft
<b>Number of Poles:</b>	1
<b>Pole Spacing:</b>	0

## Foundation Specifications

<b>Foundation Type:</b>	Round
<b>Foundation Dimensions:</b>	Ø36 in
<b>Foundation Depth (below grade):</b>	Pile 1: 30.00 ft
<b>Foundation Volume:</b>	7.854 y <sup>3</sup>

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	B
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	94 West St, Easthampton, MA 01027, USA
<b>Wind Speed:</b>	114 mph
<b>Snow Load:</b>	40 psf

### **Design Disclaimer**

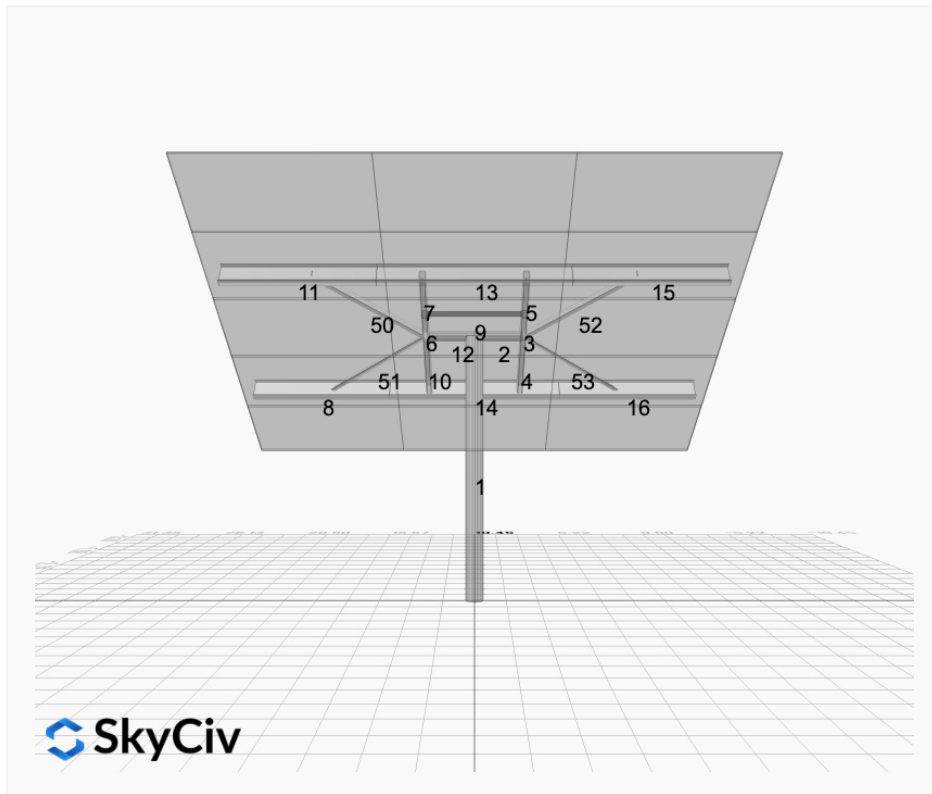
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

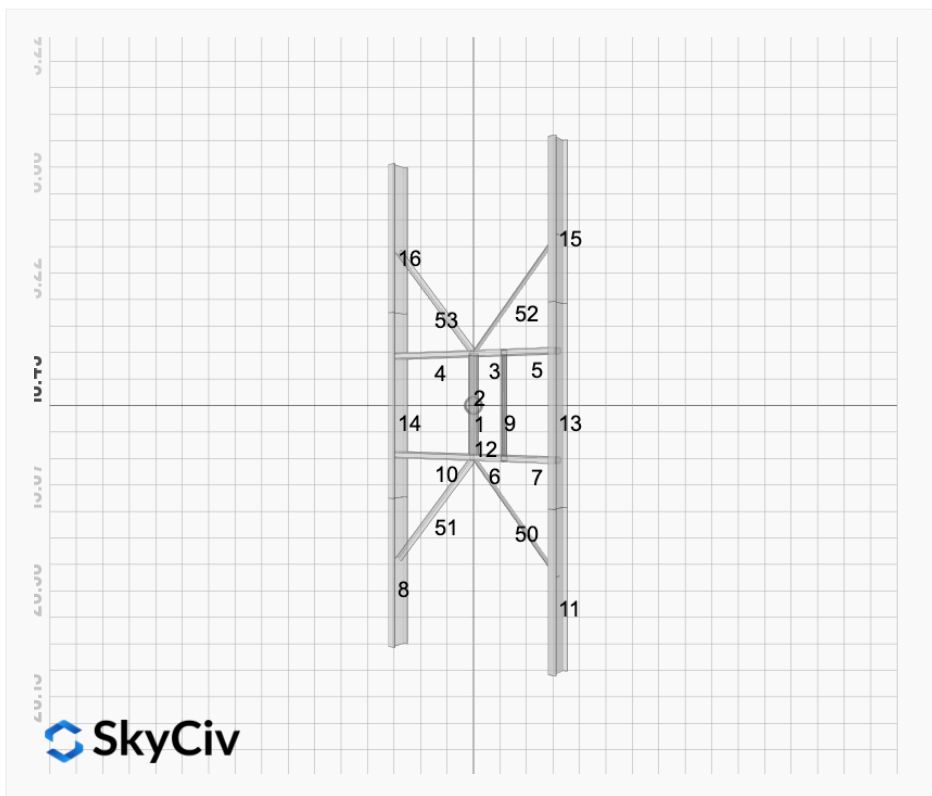
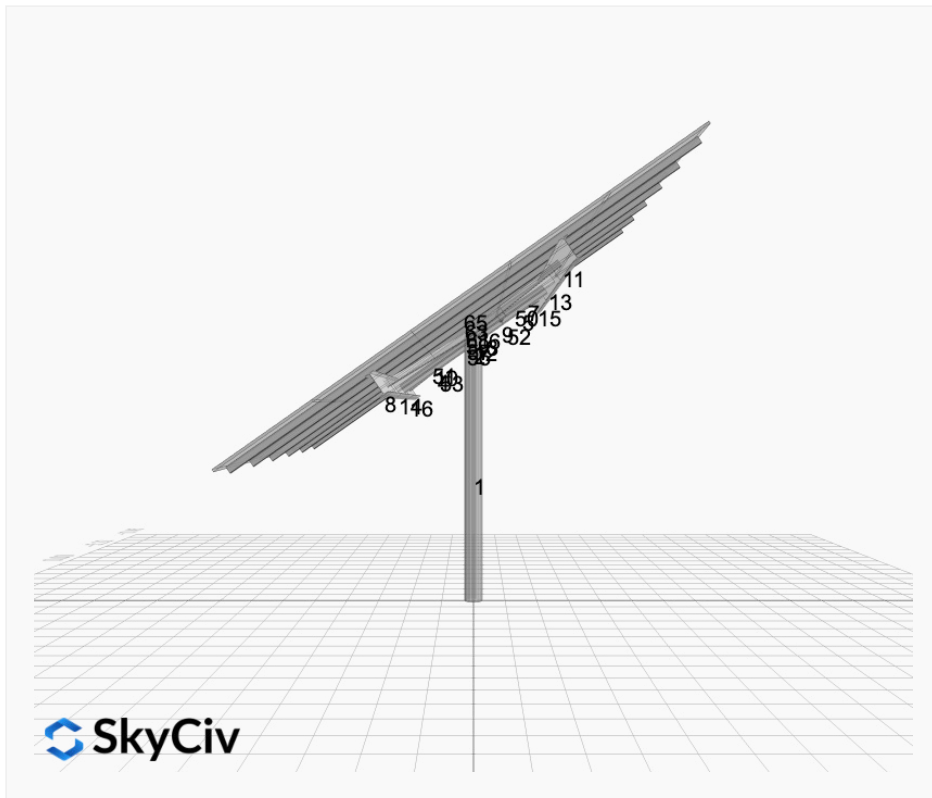
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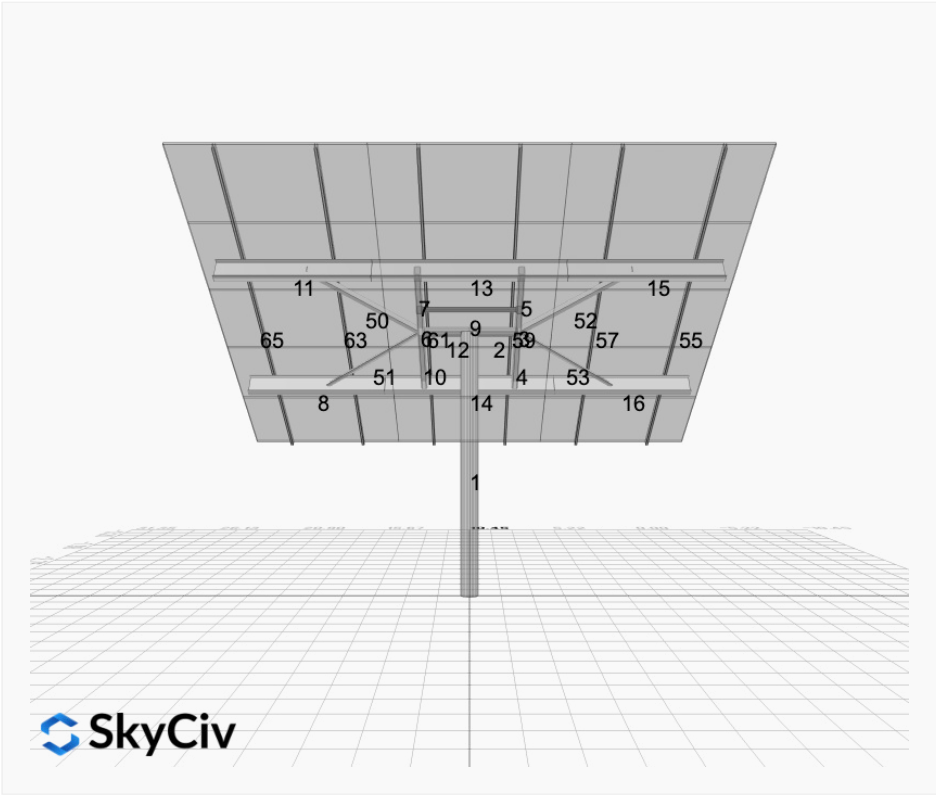
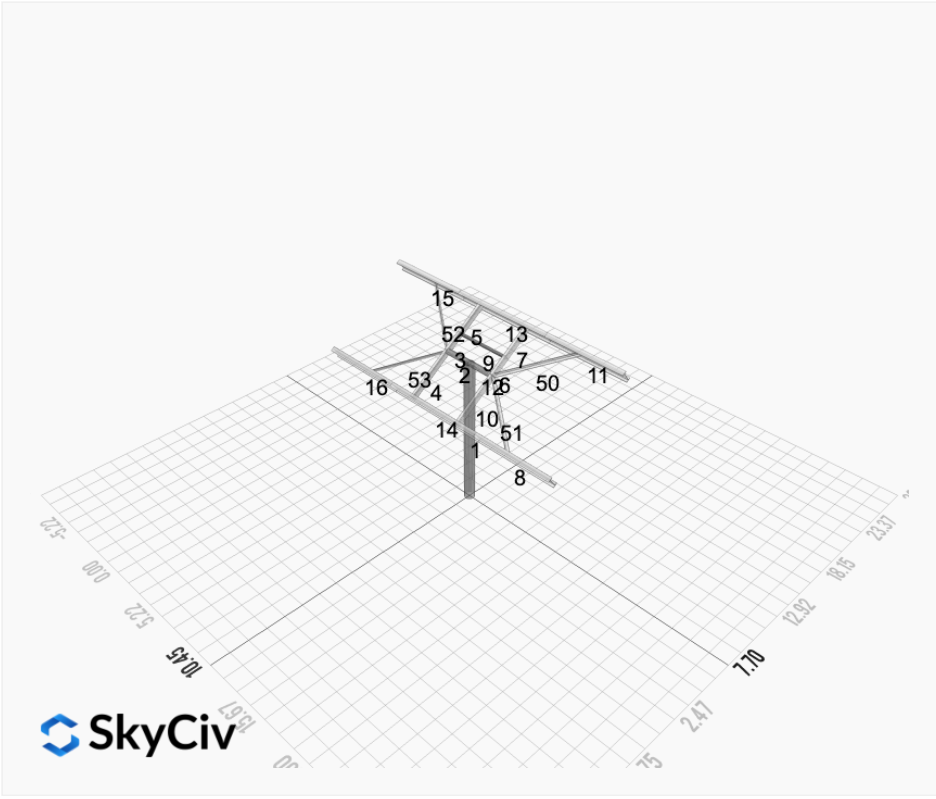
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## Design Notes:

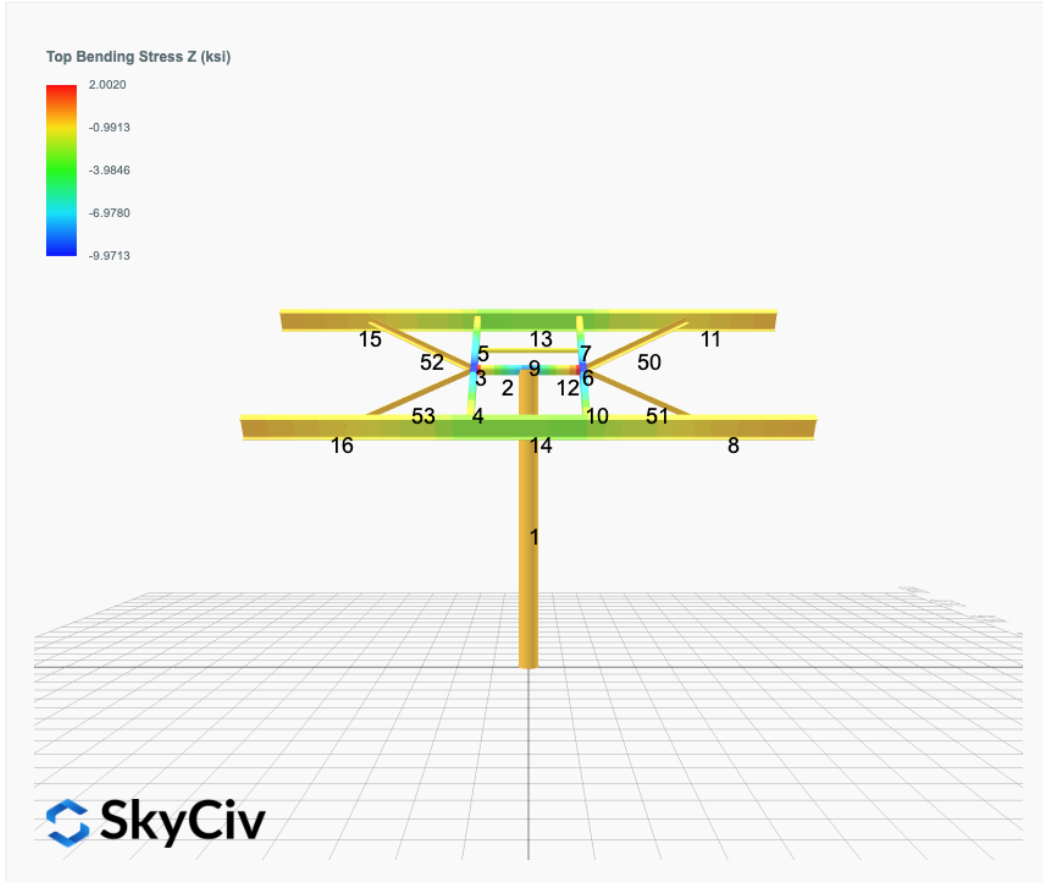
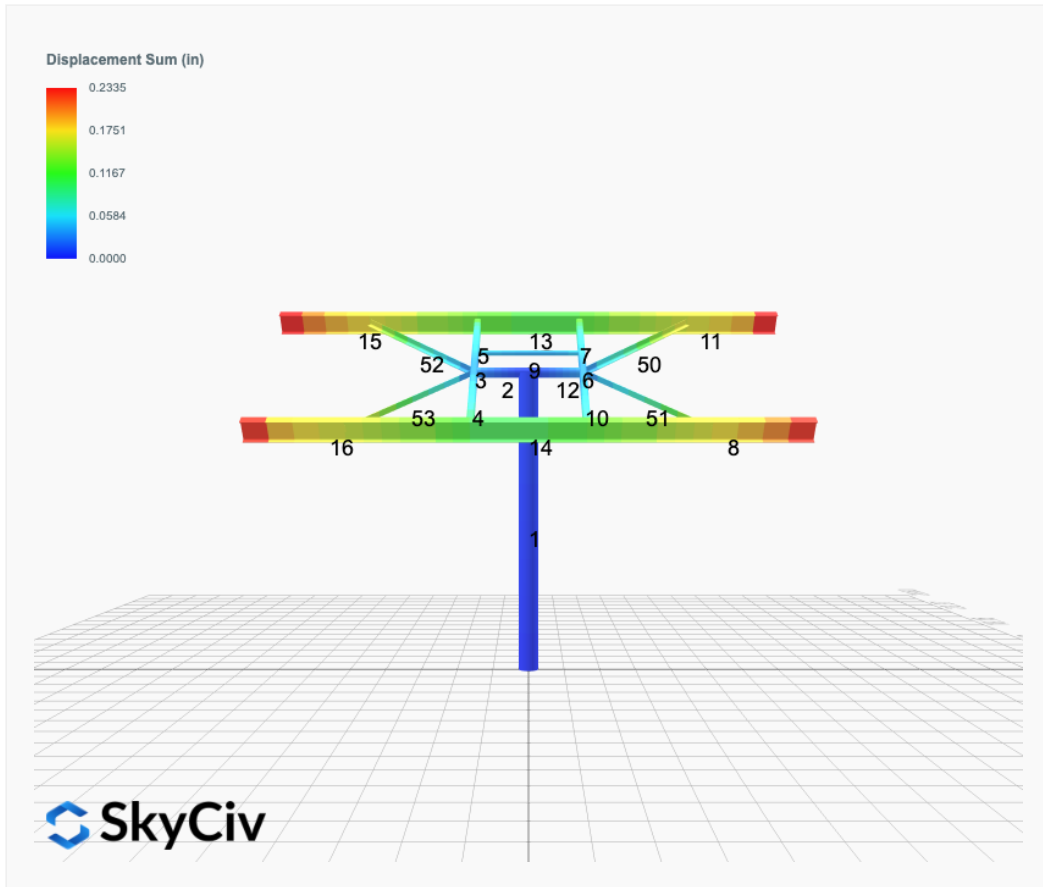
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



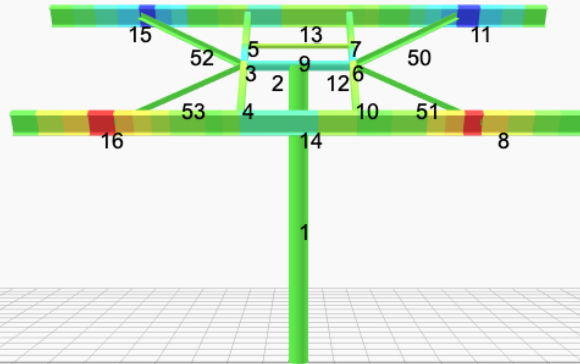
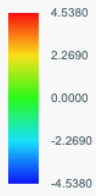




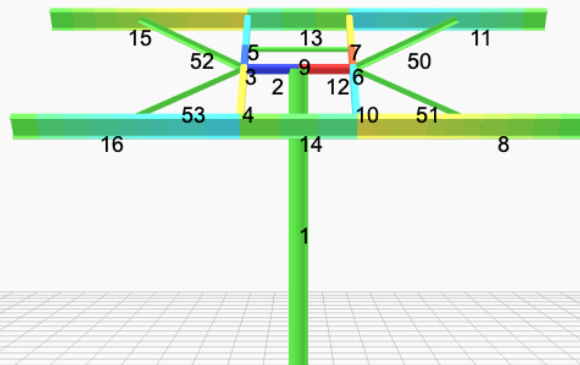
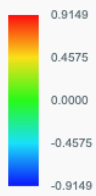
# FEM Results (Envelope Worst Case for each member)



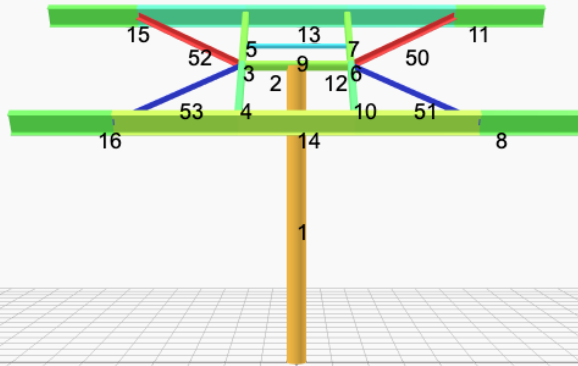
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.8738	0.0000	-0.0000	-0.0000	0.0310
ULS: 2. D + L	0.0000	2.8738	0.0000	-0.0000	-0.0000	0.0310
ULS: 3. D + (S or Lr or R)	0.0000	7.4959	0.0000	-0.0000	-0.0000	0.0596
ULS: 3. D + (S or Lr or R)	0.0000	2.8738	0.0000	-0.0000	-0.0000	0.0310
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.3403	0.0000	-0.0000	-0.0000	0.0524
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.8738	0.0000	-0.0000	-0.0000	0.0310
ULS: 5b. D + 0.7E	0.0000	2.8738	0.0000	-0.0000	-0.0000	0.0310
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.3403	0.0000	-0.0000	-0.0000	0.0524
ULS: 8. 0.6D + 0.7E	0.0000	1.7243	0.0000	-0.0000	-0.0000	0.0186
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.9585	8.5270	0.0000	-0.0000	-0.0000	44.6954
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.9585	8.5270	0.0000	-0.0000	-0.0000	44.6954
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3400	-1.8962	0.0000	-0.0000	-0.0000	-35.5792
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.7833	-1.1012	0.0000	-0.0000	-0.0000	-40.7725
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.9688	10.5803	0.0000	-0.0000	-0.0000	33.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.9688	10.5803	0.0000	-0.0000	-0.0000	33.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5050	2.7629	0.0000	-0.0000	-0.0000	-26.6552
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0875	3.3591	0.0000	-0.0000	-0.0000	-30.5501
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.9688	7.1137	0.0000	-0.0000	-0.0000	33.5293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.9688	7.1137	0.0000	-0.0000	-0.0000	33.5293
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.5050	-0.7037	0.0000	-0.0000	-0.0000	-26.6766
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.0875	-0.1074	0.0000	-0.0000	-0.0000	-30.5716
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.9585	7.3775	0.0000	-0.0000	-0.0000	44.6830
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.9585	7.3775	0.0000	-0.0000	-0.0000	44.6830
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3400	-3.0457	0.0000	-0.0000	-0.0000	-35.5916
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.7833	-2.2507	0.0000	-0.0000	-0.0000	-40.7849

### 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	15.5549
Shear X	-6.5974
Shear Z	0.0000
Moment X	0.0013
Moment Y (Twist)	0.0017
Moment Z	75.8456

### Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	10.5803
Shear X	-3.9585
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	44.6954

# Project Details

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

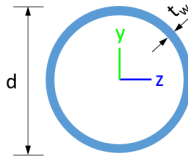


# Design Input Information

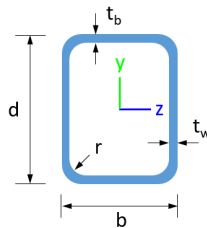
Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F <sub>y</sub> (ksi)	F <sub>u</sub> (ksi)
1	29000	50	65

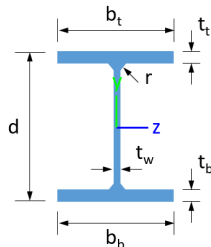
## Section Dimensions



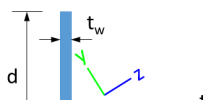
ID	Name	d (in)	t <sub>w</sub> (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



ID	Name	d (in)	b (in)	t <sub>w</sub> (in)	t <sub>b</sub> (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	t <sub>w</sub> (in)	b <sub>t</sub> (in)	b <sub>b</sub> (in)	t <sub>t</sub> (in)	t <sub>b</sub> (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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	377.97	199.68	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	86.08	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	86.08	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.15	6.46	56.26	44.91
14	159.30	97.43	30.99	6.46	56.26	44.91
15	159.30	86.08	46.90	6.46	56.26	44.91
16	159.30	86.08	46.90	6.46	56.26	44.91
50	41.27	8.45	1.63	0.76	15.23	10.15
51	41.27	8.45	1.63	0.76	15.23	10.15
52	41.27	8.45	1.63	0.76	15.23	10.15
53	41.27	8.45	1.63	0.76	15.23	10.15

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	$\delta$	Status
1	0.078	0.911	0.000	0.058	0.000	0.949	#13	0.467	Not Required	Pass
2	0.005	0.470	0.240	0.101	0.044	0.712	#13	0.054	Not Required	Pass
3	0.003	0.744	0.111	0.074	0.046	0.802	#13	0.046	Not Required	Pass
4	0.003	0.724	0.039	0.072	0.005	0.744	#13	0.082	Not Required	Pass
5	0.003	0.462	0.022	0.074	0.008	0.473	#13	0.076	Not Required	Pass
6	0.003	0.744	0.111	0.074	0.046	0.802	#13	0.046	Not Required	Pass
7	0.003	0.462	0.022	0.074	0.008	0.473	#13	0.076	Not Required	Pass
8	0.014	0.153	0.140	0.042	0.012	0.194	#21	0.459	Not Required	Pass
9	0.021	0.072	0.051	0.001	0.000	0.128	#13	0.137	Not Required	Pass
10	0.003	0.723	0.039	0.072	0.005	0.744	#13	0.082	Not Required	Pass
11	0.008	0.157	0.140	0.044	0.012	0.191	#21	0.306	Not Required	Pass
12	0.005	0.471	0.240	0.101	0.044	0.712	#13	0.054	Not Required	Pass
13	0.008	0.411	0.028	0.056	0.007	0.427	#13	0.204	Not Required	Pass
14	0.013	0.406	0.025	0.055	0.007	0.417	#13	0.306	Not Required	Pass
15	0.008	0.157	0.140	0.044	0.012	0.191	#21	0.306	Not Required	Pass
16	0.014	0.153	0.140	0.042	0.012	0.194	#21	0.306	Not Required	Pass
50	0.191	0.010	0.002	0.002	0.001	0.201	#21	0.783	Not Required	Pass
51	0.038	0.006	0.016	0.002	0.002	0.055	#21	0.522	Not Required	Pass
52	0.191	0.010	0.002	0.002	0.001	0.201	#24	0.783	Not Required	Pass
53	0.038	0.006	0.016	0.002	0.002	0.055	#24	0.522	Not Required	Pass

Definitions

$\Phi_t$  Safety factor for tensile

$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	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
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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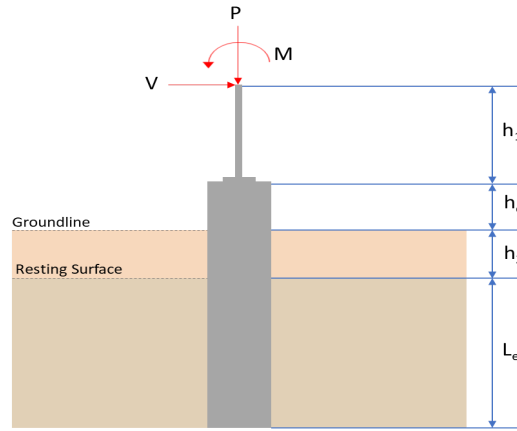
## SkyCiv Foundation Design

Pile Foundation

### Design Information :

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

### Pile Input



### Geometry

Pile shape: round

$D = 36$  in - Pile diameter

$L = 30$  ft - Total pile length

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

$h_2 = 0$  ft - Depth to resisting surface

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

### Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

### Tabulation of Loads

Load Component	ASD	LRFD
$P$ (kip)	10.580	15.555
$V_x$ (kip)	-3.958	-6.597
$V_z$ (kip)	0.000	0.000
$M_x$ (kipft)	0.000	0.001
$M_z$ (kipft)	44.695	75.846

### Material Properties

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

### Required depth to resist lateral loads (ASD)

$H$  - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

### Considering x-direction:

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-3.958 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3193 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(44.695 \text{ kipft}) + ((-3.958 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 14.898 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.0648 \text{ ft}$  - 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,

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

$$L_{e,req} = \text{MAX}[(9.0648 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 9.065 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (30 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 30 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(9.065 \text{ ft})}{(30 \text{ ft})}$$

$$\text{Ratio} = 0.30217$$

Status: **PASS**  
Ratio: **0.300**

### 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

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

$$q = \frac{(10.58 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.4968 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.4968 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.74838$$

Status: **PASS**  
Ratio: **0.750**

Czerniak

### Lateral Soil Pressure (ASD):

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(30 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 10$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -1.3193 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 14.898 \text{ kipft/ft}$  - Overturning moment per length of pile,

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

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

$$a = \frac{(4 \times (14.898 \text{ kipft/ft}) \times (30 \text{ ft})) + (3 \times (-1.3193 \text{ kip/ft}) \times (30 \text{ ft})^2)}{(6 \times (14.898 \text{ kipft/ft})) + (4 \times (-1.3193 \text{ kip/ft}) \times (30 \text{ ft}))}$$

$$a = 21.598 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (14.898 \text{ kipft/ft})) + (3 \times (-1.3193 \text{ kip/ft}) \times (30 \text{ ft}))]^2}{(30 \text{ ft})^2 \times [(3 \times (14.898 \text{ kipft/ft})) + (2 \times (-1.3193 \text{ kip/ft}) \times (30 \text{ ft}))]}$$

$$p = -0.13286 \text{ kip/ft}^2$$

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

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

$$s = \frac{9.425 \times [(2 \times (14.898 \text{ kipft/ft})) + ((-1.3193 \text{ kip/ft}) \times (30 \text{ ft}))]}{(30 \text{ ft})^2}$$

$$s = -0.10245 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(21.598 \text{ ft})}{2}$$

$$p_a = 1.6198 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.13286 \text{ kip/ft}^2)}{(1.6198 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.082019$$

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

$$p_s = R L_c$$

$$p_s = (150 \text{ psf/ft}) \times (30 \text{ ft})$$

$$p_s = 4.5 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

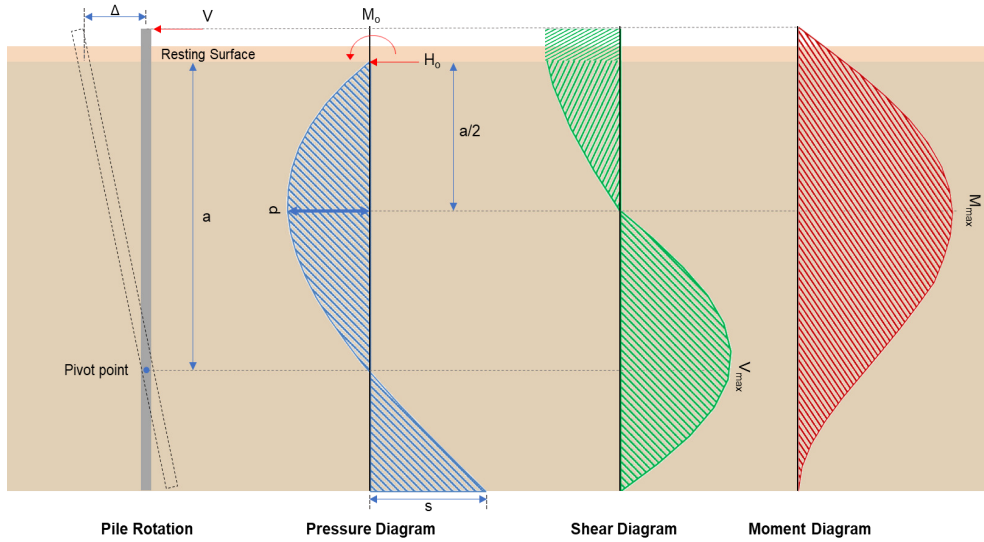
$$(-0.10245 \text{ kip/ft}^2)$$

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

$$\text{Ratio} = \frac{-0.022767}{(4.5 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.022767$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-6.597 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -2.199 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(75.846 \text{ kipft}) + ((-6.597 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 25.282 \text{ kipft/ft}$$

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

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

$$E = \frac{(25.282 \text{ kipft/ft})}{(-2.199 \text{ kip/ft})}$$

$$E = 11.497 \text{ ft}$$

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

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

$$a = \frac{(4 \times (25.282 \text{ kipft/ft}) \times (30 \text{ ft})) + (3 \times (-2.199 \text{ kip/ft}) \times (30 \text{ ft})^2)}{(6 \times (25.282 \text{ kipft/ft})) + (4 \times (-2.199 \text{ kip/ft}) \times (30 \text{ ft}))}$$

$$a = 21.587 \text{ ft}$$

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

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

$$V_{max} = ((-2.199 \text{ kip/ft}) \times (36 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (11.497 \text{ ft})}{(30 \text{ ft})} + 3 \right) \times \left( \frac{(21.587 \text{ ft})}{(30 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (11.497 \text{ ft})}{(30 \text{ ft})} + 2 \right) \times \left( \frac{(21.587 \text{ ft})}{(30 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.8871 \text{ kip}$$

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

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

$$M_{max} = ((-2.199 \text{ kip/ft}) \times (36 \text{ in}) \times (30 \text{ ft})) \times \left[ \left( \frac{(11.497 \text{ ft})}{(30 \text{ ft})} + \frac{(21.587 \text{ ft})}{2 \times (30 \text{ ft})} \right) - \left[ \left( \frac{4 \times (11.497 \text{ ft})}{(30 \text{ ft})} + 3 \right) \times \left( \frac{(21.587 \text{ ft})}{2 \times (30 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (11.497 \text{ ft})}{(30 \text{ ft})} + 2 \right) \times \left( \frac{(21.587 \text{ ft})}{2 \times (30 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 115.72 \text{ kipft}$$

### 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,

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

$$a = \frac{(4 \times (0.00033333 \text{ kipft/ft}) \times (30 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (30 \text{ ft})^2)}{(6 \times (0.00033333 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (30 \text{ ft}))}$$

$$a = 20 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left( \frac{(0.00033333 \text{ kipft/ft}) \times (36 \text{ in})}{(30 \text{ ft})} \right) \times \left( \frac{(20 \text{ ft})}{(30 \text{ ft})} - 1 \right) \times \left( \frac{(20 \text{ ft})}{(30 \text{ ft})} \right)^2$$

$$V_{max} = 0.000059259 \text{ kip}$$

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

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

$$M_{max} = ((0.00033333 \text{ kipft/ft}) \times (36 \text{ in})) \times \left[ 1 - \left( 4 \times \frac{(20 \text{ ft})}{2 \times (30 \text{ ft})} \right)^3 + \left( 3 \times \frac{(20 \text{ ft})}{2 \times (30 \text{ ft})} \right)^4 \right]$$

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

### 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

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

$$A_{st,required} = Min \left[ \frac{\frac{(15.555 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.887 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-36.887 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

$n_{rebar}$  - 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  $\phi P_N$  - Allowable axial compressive strength

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

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

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

### Axial Compression Strength (ACI 318-19, LRFD)

Ratio - Capacity

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

$$Ratio = \frac{(15.555 \text{ kip})}{(1253.9 \text{ kip})}$$

$$Ratio = 0.012405$$

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

### Shear Strength (ACI 318-19, LRFD)

#### Parameters:

$b_w = 36$  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  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5$  ksi  $\rightarrow$  2500 psi,

22.5.5.1.1  $V_{c,max}$  - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5$  ksi  $\rightarrow$  2500 psi,  $P = 15.555$  kip  $\rightarrow$  15555 lbf,

22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(15555 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.079 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5$  ksi  $\rightarrow$  2500 psi,

22.5.5.1.2  $V_{c,b}$  - 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

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

$$V_c = MIN [(186.09 \text{ kip}), (77.079 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.079 \text{ kip}$$

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 2.5$  ksi  $\rightarrow$  2500 psi,

22.5.5.1.2  $V_{s,a}$  - Shear strength of steel (a)

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

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

$A_v$  - Ties rebar area,

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

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

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

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

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  $\phi V_n$  - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.079 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.912 \text{ kip}$$

**Considering x-direction:**

$V_{max} = 8.8871 \text{ kip}$  - Maximum shear force in the x-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(8.8871 \text{ kip})}{(74.912 \text{ kip})}$$

$$\text{Ratio} = 0.11863$$

Status: **PASS**  
Ratio: **0.120**

**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  $\phi M_{n,2}$

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

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

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

Therefore,  
 $\phi M_n$  - 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:**

$M_{max} = 115.72 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(115.72 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 1.8656$$

Status: **FAIL**  
Ratio: **1.870**

**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**