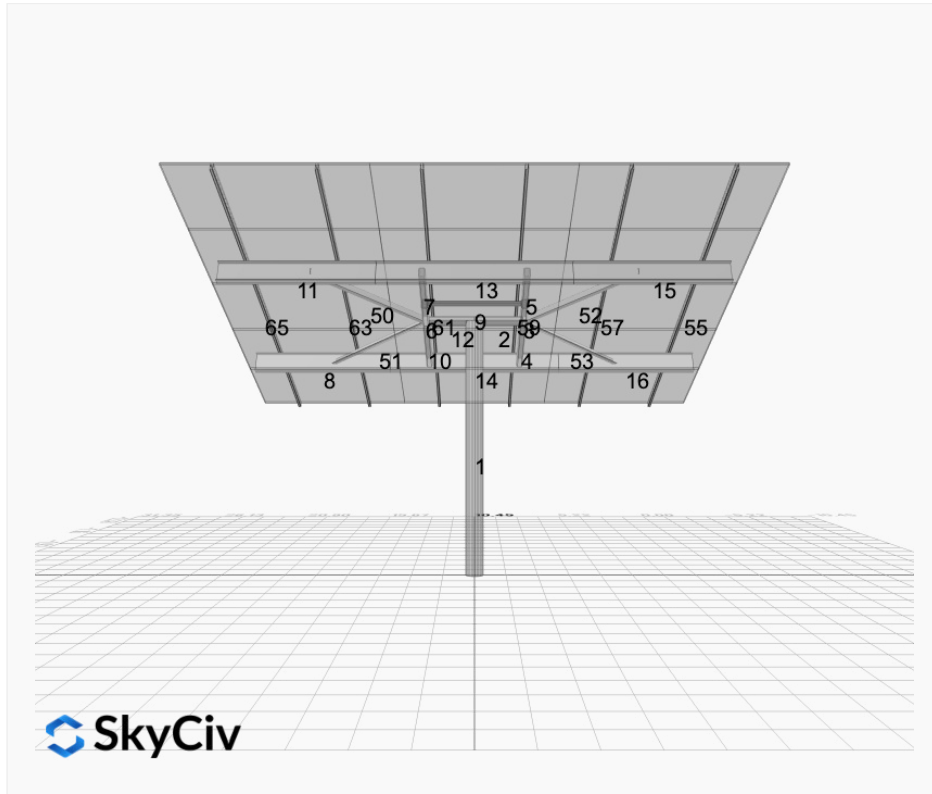


Project Name: Tiskus - Easthampton DE RE 25deg 6ft Struts - V1Jb
Location: 94 West St, Easthampton, MA 01027, USA
Unique ID: 1P-0-8TOP-XD-72-L-5Hx3W-STRUTS-7L0D
Dealer: _____

Date: Wed May 14 2025
Number of Modules: 15
Number of Poles: 1
Date Sold: _____



Array Dimensions N/S	18.80 ft
Array Dimensions E/W	20.90 ft
Winter Tilt Angle	25
Front Edge Clearance	6.5 ft

MT Solar Bill of Materials (1P-0-8TOP-XD-72-L-5Hx3W-STRUTS-7L0D)

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

Duty Classification:	XD
Module Width:	44.61 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Winter Tilt Angle:	25
Front Edge Clearance:	6.5
Total Array Height at Tilt:	14.44 ft
Total Frame Length:	19.50 ft
Module Info/Notes:	SILFAB SIL-530 XM
Array Dimensions N/S:	18.80 ft
Array Dimensions E/W:	20.90 ft
Rail Length:	225.55 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	10.47 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 9.50 ft
Foundation Volume:	2.487 y ³

Site Info

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

Design Disclaimer

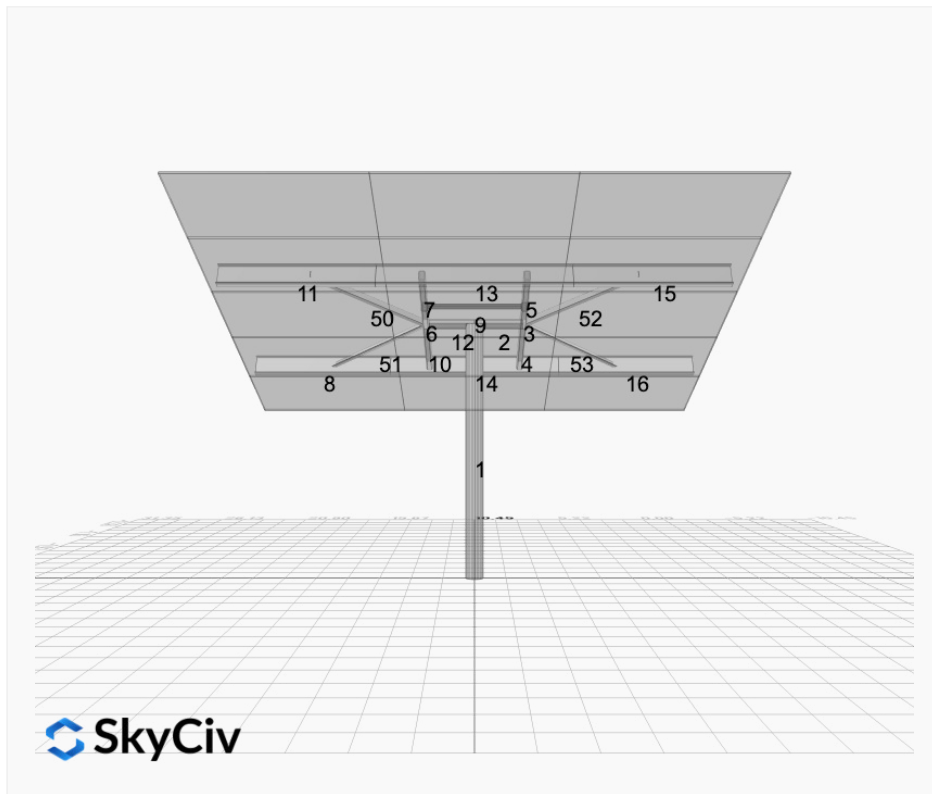
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

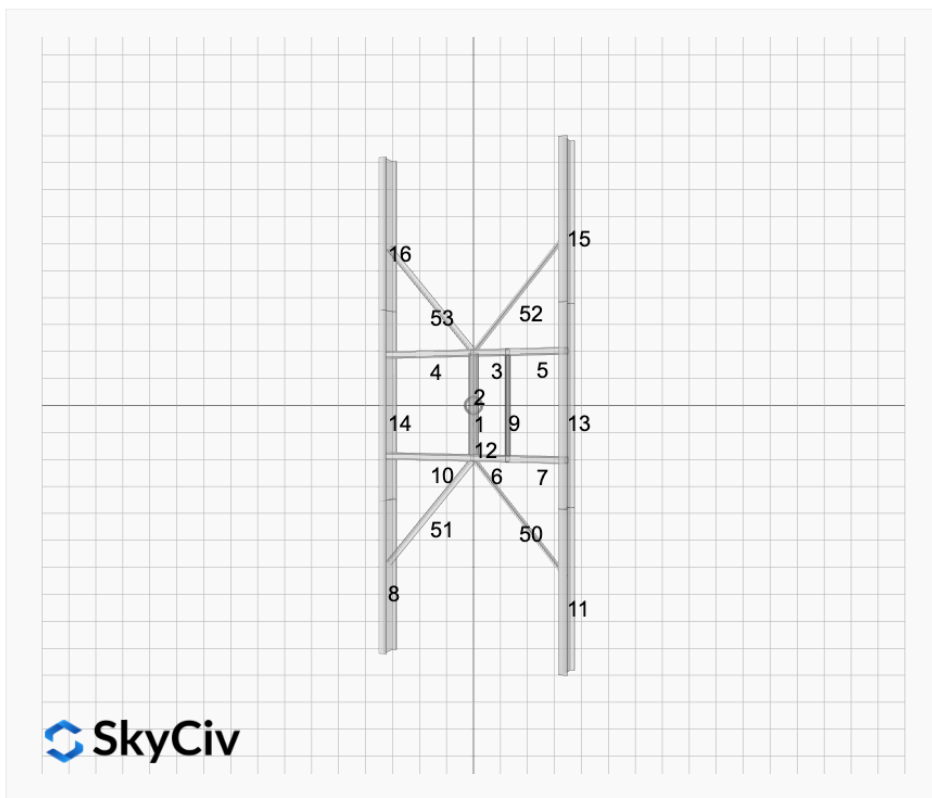
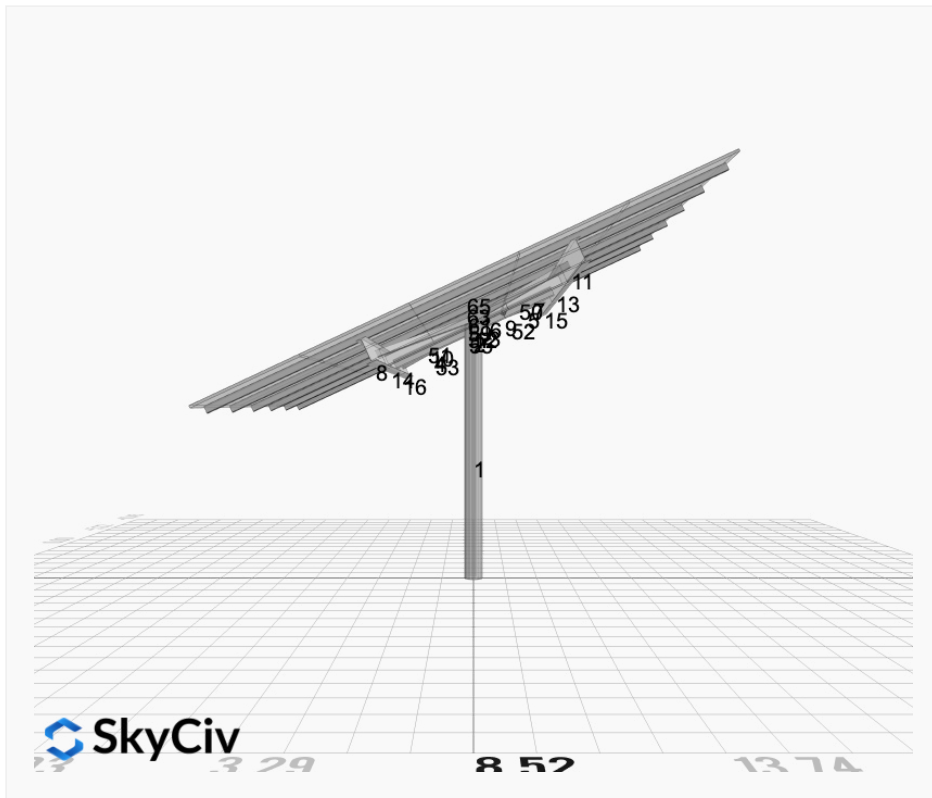
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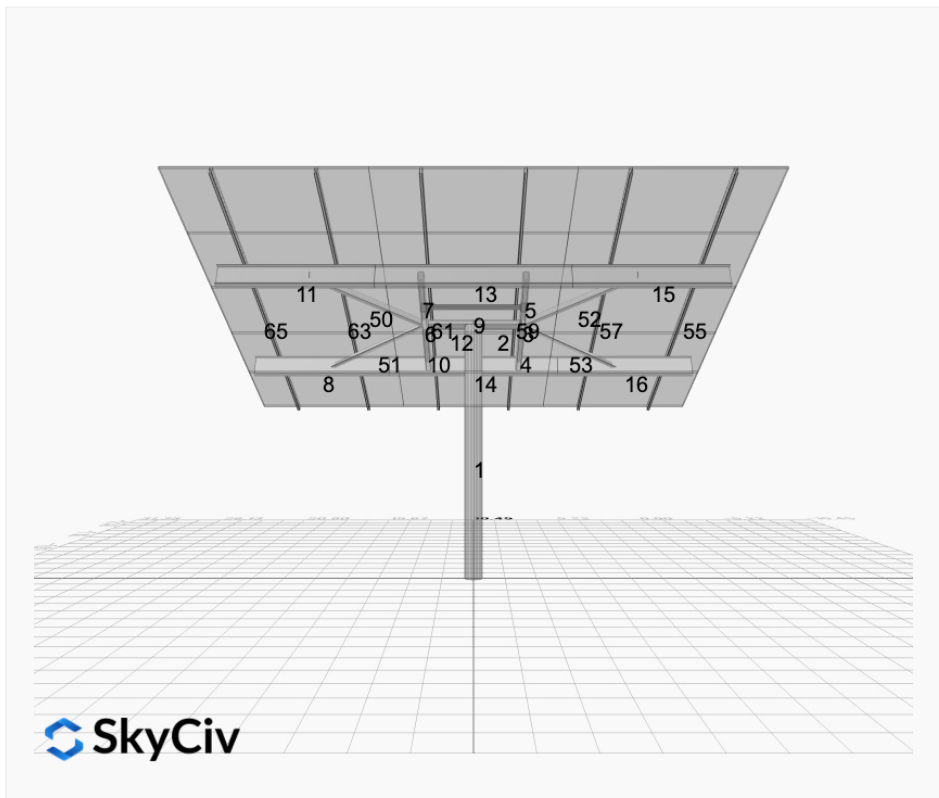
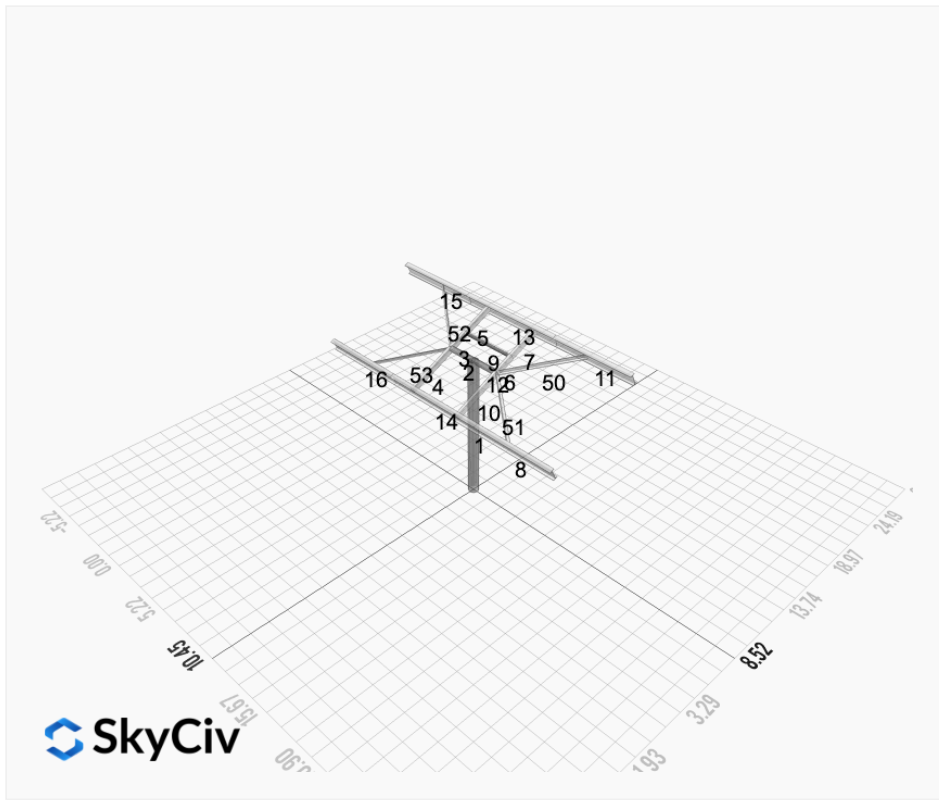
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Design Notes:

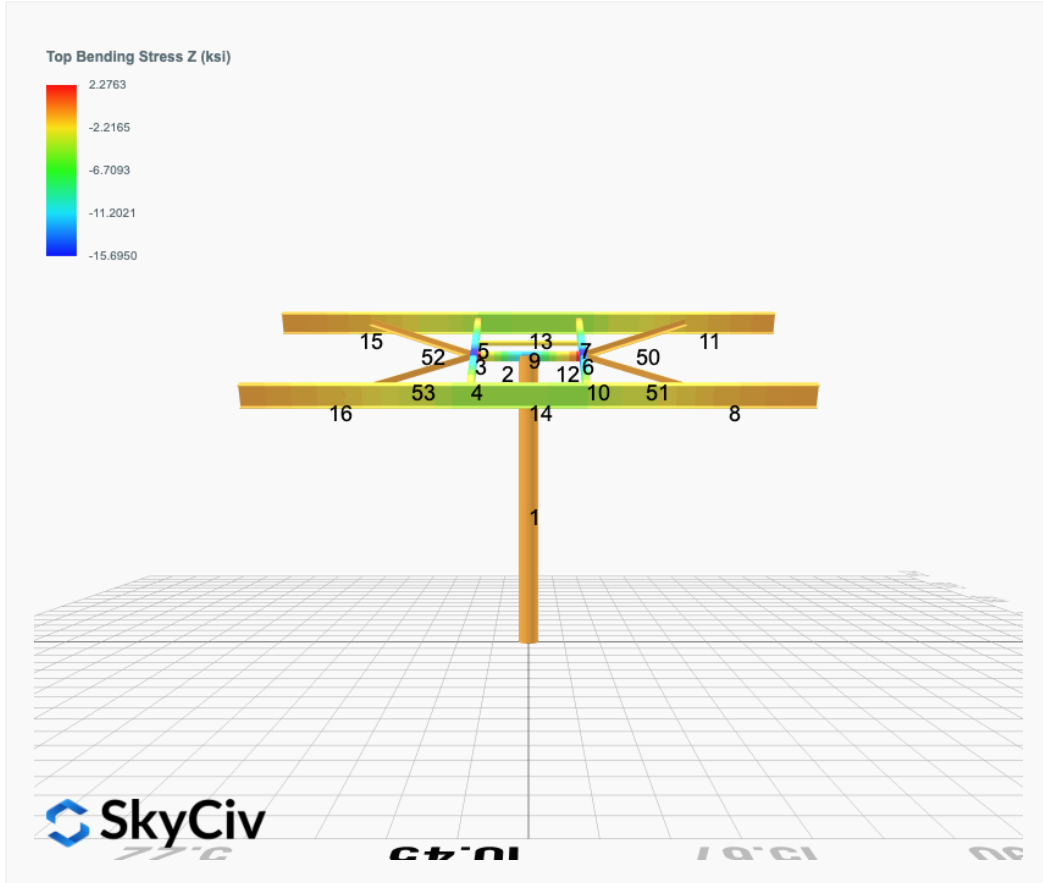
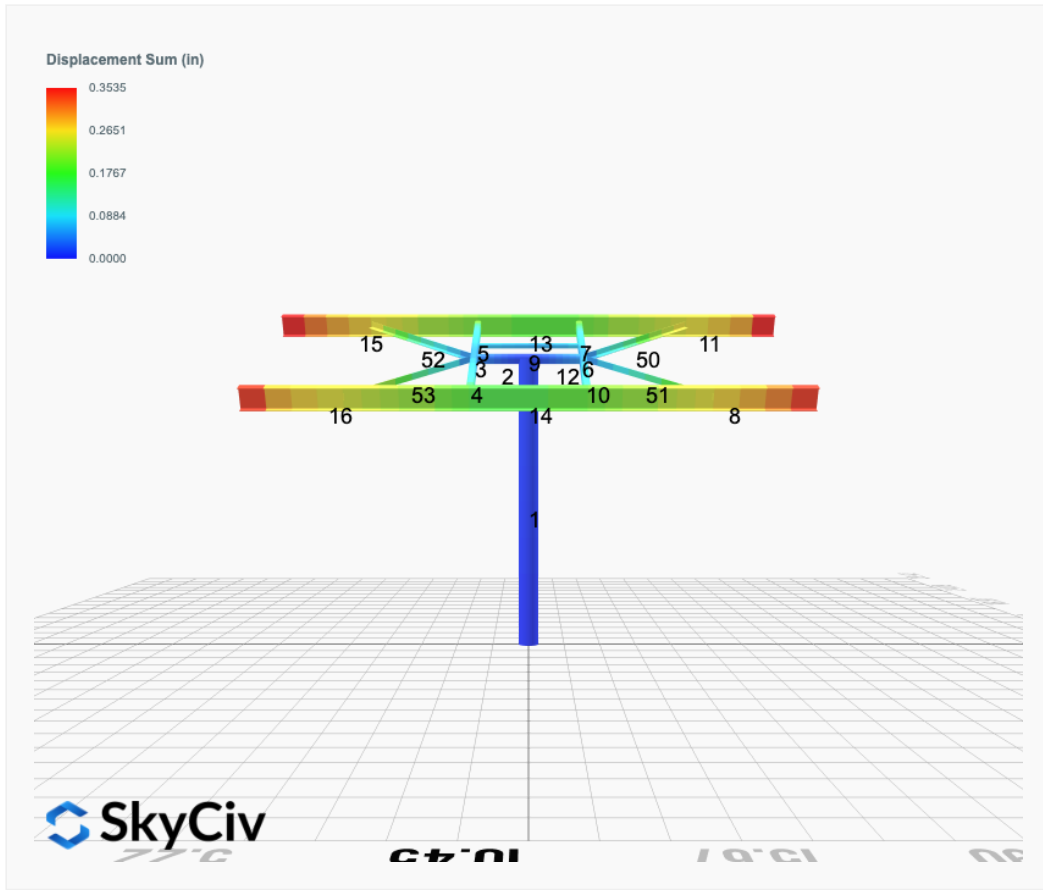
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigned are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel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



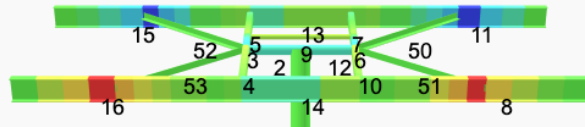




FEM Results (Envelope Worst Case for each member)



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	2.8618	0.0000	-0.0000	-0.0000	0.0329
ULS: 2. D + L	0.0000	2.8618	0.0000	-0.0000	-0.0000	0.0329
ULS: 3. D + (S or Lr or R)	0.0000	9.4368	0.0000	-0.0000	-0.0000	0.0805
ULS: 3. D + (S or Lr or R)	0.0000	2.8618	0.0000	-0.0000	-0.0000	0.0329
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	7.7930	0.0000	-0.0000	-0.0000	0.0686
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.8618	0.0000	-0.0000	-0.0000	0.0329
ULS: 5b. D + 0.7E	0.0000	2.8618	0.0000	-0.0000	-0.0000	0.0329
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	7.7930	0.0000	-0.0000	-0.0000	0.0686
ULS: 8. 0.6D + 0.7E	0.0000	1.7171	0.0000	-0.0000	-0.0000	0.0198
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5521	8.3347	0.0000	-0.0000	-0.0000	27.8037
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5521	8.3347	0.0000	-0.0000	-0.0000	27.8037
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2331	-1.9270	0.0000	-0.0000	-0.0000	-22.5021
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.9140	-1.2429	0.0000	-0.0000	-0.0000	-31.8944
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9140	11.8977	0.0000	-0.0000	-0.0000	20.8966
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9140	11.8977	0.0000	-0.0000	-0.0000	20.8966
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6748	4.2014	0.0000	-0.0000	-0.0000	-16.8327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4355	4.7145	0.0000	-0.0000	-0.0000	-23.8769
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9140	6.9665	0.0000	-0.0000	-0.0000	20.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.9140	6.9665	0.0000	-0.0000	-0.0000	20.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6748	-0.7298	0.0000	-0.0000	-0.0000	-16.8683
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4355	-0.2167	0.0000	-0.0000	-0.0000	-23.9126
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5521	7.1900	0.0000	-0.0000	-0.0000	27.7905
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5521	7.1900	0.0000	-0.0000	-0.0000	27.7905
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2331	-3.0717	0.0000	-0.0000	-0.0000	-22.5153
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.9140	-2.3876	0.0000	-0.0000	-0.0000	-31.9076

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	18.5149
Shear X	-4.2534
Shear Z	0.0000
Moment X	0.0007
Moment Y (Twist)	0.0013
Moment Z	54.2604

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	11.8977
Shear X	-2.5521
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	31.9076

Project Details

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

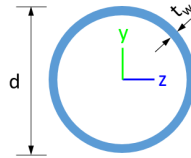


Design Input Information

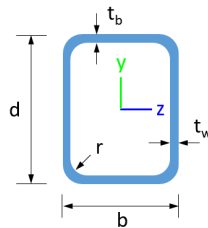
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

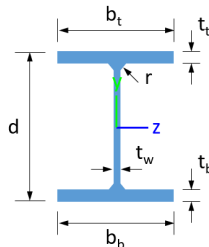
Section Dimensions



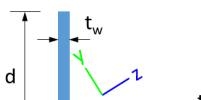
ID	Name	d (in)	t _w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30



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	209.52	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	31.18	6.46	56.26	44.91
14	159.30	97.43	30.74	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

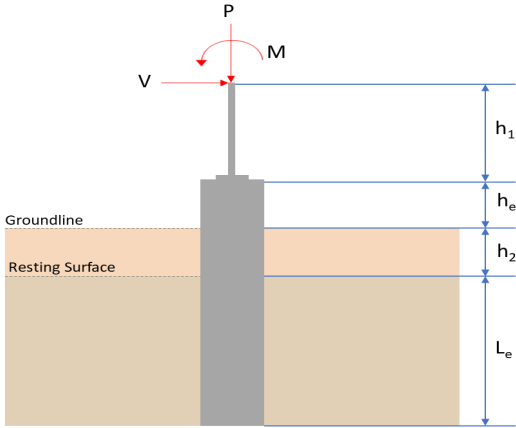
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.088	0.651	0.000	0.038	0.000	0.652	#16	0.449	Not Required	Pass
2	0.005	0.575	0.162	0.121	0.028	0.695	#21	0.054	Not Required	Pass
3	0.003	0.806	0.106	0.081	0.044	0.914	#21	0.046	Not Required	Pass
4	0.003	0.790	0.037	0.079	0.004	0.829	#21	0.082	Not Required	Pass
5	0.003	0.500	0.021	0.080	0.008	0.522	#21	0.076	Not Required	Pass
6	0.003	0.806	0.106	0.081	0.044	0.914	#21	0.046	Not Required	Pass
7	0.003	0.500	0.021	0.080	0.008	0.522	#21	0.076	Not Required	Pass
8	0.013	0.167	0.136	0.046	0.011	0.199	#21	0.459	Not Required	Pass
9	0.020	0.086	0.043	0.001	0.000	0.140	#21	0.137	Not Required	Pass
10	0.003	0.790	0.037	0.079	0.004	0.829	#21	0.082	Not Required	Pass
11	0.007	0.170	0.136	0.047	0.012	0.196	#21	0.306	Not Required	Pass
12	0.005	0.575	0.162	0.121	0.028	0.695	#21	0.054	Not Required	Pass
13	0.007	0.445	0.027	0.061	0.006	0.475	#21	0.204	Not Required	Pass
14	0.013	0.444	0.024	0.060	0.006	0.466	#21	0.306	Not Required	Pass
15	0.007	0.170	0.136	0.047	0.012	0.196	#21	0.306	Not Required	Pass
16	0.013	0.167	0.136	0.046	0.011	0.199	#21	0.306	Not Required	Pass
50	0.184	0.010	0.001	0.002	0.001	0.194	#21	0.783	Not Required	Pass
51	0.037	0.007	0.015	0.002	0.002	0.054	#21	0.522	Not Required	Pass
52	0.184	0.010	0.001	0.002	0.001	0.194	#24	0.783	Not Required	Pass
53	0.037	0.007	0.015	0.002	0.002	0.054	#24	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 $L = 9.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 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>11.898</td> <td>18.515</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.552</td> <td>-4.253</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>31.908</td> <td>54.260</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)	11.898	18.515	V_x (kip)	-2.552	-4.253	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.001	M_z (kipft)	31.908	54.260	
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																									
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V_z (kip)	0.000	0.000																										
M_x (kipft)	0.000	0.001																										
M_z (kipft)	31.908	54.260																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.552 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.85067 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(31.908 \text{ kipft}) + ((-2.552 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.636 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.636 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(8.636 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 8.636 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (9.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 9.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(8.636 \text{ ft})}{(9.5 \text{ ft})}$$

$$Ratio = 0.90905$$

Status: **PASS**
Ratio: **0.910**

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

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

$$q = \frac{(11.898 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.6832 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(1.6832 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.84161$$

Status: **PASS**
Ratio: **0.840**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3.1667$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.85067 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 10.636 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.636 \text{ kipft/ft}) \times (9.5 \text{ ft})) + (3 \times (-0.85067 \text{ kip/ft}) \times (9.5 \text{ ft})^2)}{(6 \times (10.636 \text{ kipft/ft})) + (4 \times (-0.85067 \text{ kip/ft}) \times (9.5 \text{ ft}))}$$

$$a = 6.5995 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (10.636 \text{ kipft/ft})) + (3 \times (-0.85067 \text{ kip/ft}) \times (9.5 \text{ ft}))]^2}{(9.5 \text{ ft})^2 \times [(3 \times (10.636 \text{ kipft/ft})) + (2 \times (-0.85067 \text{ kip/ft}) \times (9.5 \text{ ft}))]}$$

$$p = 0.27762 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_c ,

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

$$s = \frac{9.425 \times [(2 \times (10.636 \text{ kipft/ft})) + ((-0.85067 \text{ kip/ft}) \times (9.5 \text{ ft}))]}{(9.5 \text{ ft})^2}$$

$$s = 1.3775 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.5995 \text{ ft})}{2}$$

$$p_a = 0.49496 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27762 \text{ kip/ft}^2)}{(0.49496 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56089$$

p_s - Allowable lateral soil pressure at depth L_c ,

$$p_s = R L_c$$

$$p_s = (150 \text{ psf/ft}) \times (9.5 \text{ ft})$$

$$p_s = 1.425 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

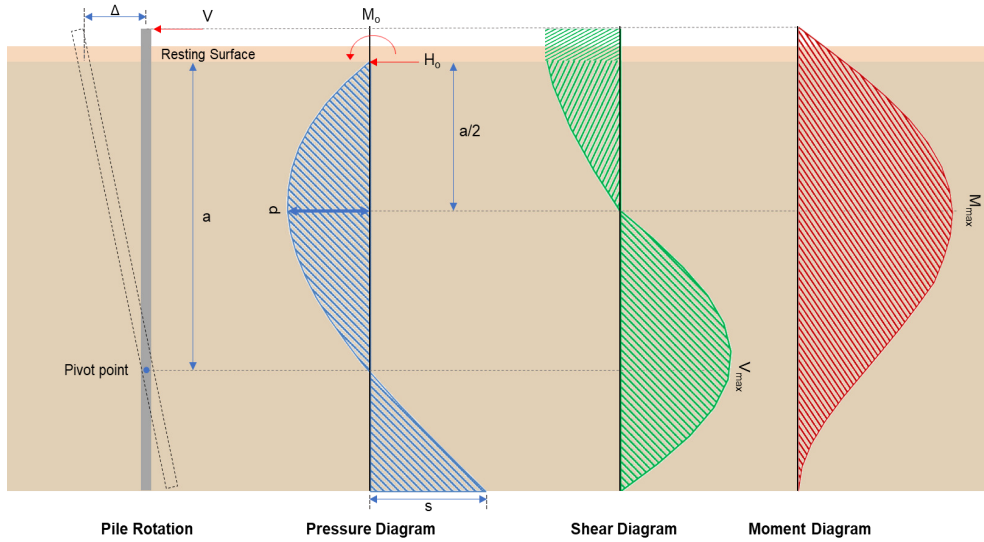
$$(1.3775 \text{ kip/ft}^2)$$

Status: **PASS**
Ratio: **0.560**

$$\text{Ratio} = \frac{\dots}{(1.425 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96669$$

Status: **PASS**
Ratio: **0.970**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.253 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.4177 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(54.26 \text{ kipft}) + ((-4.253 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 18.087 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(18.087 \text{ kipft/ft})}{(-1.4177 \text{ kip/ft})}$$

$$E = 12.758 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.087 \text{ kipft/ft}) \times (9.5 \text{ ft})) + (3 \times (-1.4177 \text{ kip/ft}) \times (9.5 \text{ ft})^2)}{(6 \times (18.087 \text{ kipft/ft})) + (4 \times (-1.4177 \text{ kip/ft}) \times (9.5 \text{ ft}))}$$

$$a = 6.596 \text{ ft}$$

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

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

$$V_{max} = ((-1.4177 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.758 \text{ ft})}{(9.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.596 \text{ ft})}{(9.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (12.758 \text{ ft})}{(9.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.596 \text{ ft})}{(9.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.911 \text{ kip}$$

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

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

$$M_{max} = ((-1.4177 \text{ kip/ft}) \times (36 \text{ in}) \times (9.5 \text{ ft})) \times \left[\left(\frac{(12.758 \text{ ft})}{(9.5 \text{ ft})} + \frac{(6.596 \text{ ft})}{2 \times (9.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.758 \text{ ft})}{(9.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.596 \text{ ft})}{2 \times (9.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (12.758 \text{ ft})}{(9.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.596 \text{ ft})}{2 \times (9.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 57.672 \text{ kipft}$$

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,

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

$$a = \frac{(4 \times (0.00033333 \text{ kipft/ft}) \times (9.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (9.5 \text{ ft})^2)}{(6 \times (0.00033333 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (9.5 \text{ ft}))}$$

$$a = 6.3333 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00033333 \text{ kipft/ft}) \times (36 \text{ in})}{(9.5 \text{ ft})} \right) \times \left(\frac{(6.3333 \text{ ft})}{(9.5 \text{ ft})} - 1 \right) \times \left(\frac{(6.3333 \text{ ft})}{(9.5 \text{ ft})} \right)^2$$

$$V_{max} = 0.00018713 \text{ kip}$$

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

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

$$M_{max} = ((0.00033333 \text{ kipft/ft}) \times (36 \text{ in})) \times \left[1 - \left(4 \times \frac{(6.3333 \text{ ft})}{2 \times (9.5 \text{ ft})} \right)^3 + \left(3 \times \frac{(6.3333 \text{ ft})}{2 \times (9.5 \text{ ft})} \right)^4 \right]$$

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

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

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

$$A_{st,required} = Min \left[\frac{\frac{(18.515 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.794 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-36.794 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

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

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

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

$$Ratio = \frac{(18.515 \text{ kip})}{(1253.9 \text{ kip})}$$

$$Ratio = 0.014766$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 18.515 \text{ kip} \rightarrow 18515 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(18515 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.581 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = MIN [(186.09 \text{ kip}), (77.581 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.581 \text{ kip}$$

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

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

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

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

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((77.581 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 75.238 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.911 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(12.911 \text{ kip})}{(75.238 \text{ kip})}$$

$$Ratio = 0.1716$$

Status: **PASS**
Ratio: **0.170**

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:

$M_{max} = 57.672 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(57.672 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.9298$$

Status: **PASS**
Ratio: **0.930**

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**