

Your Project Calculations



Project Name: Tahawus10-480

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Tahawus10-480&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=OnTnGUEPz686W8ORGakaAco3TLgySdEoVpZnCWMs4z5GpJoBqjCVR6Zj4tzBx7O5

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-HD-45-L-5Hx2W-EEE5
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	2
Total Number of Modules:	10
Desired Tilt Angle:	35
Front Edge Clearance:	4
Total Array Height at Tilt:	13.88 ft
Total Frame Length:	15.00 ft
Frame Weight:	770 lbs
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	14.70 ft
Rail Length:	208.00 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.97 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft
Foundation Volume:	3.407 y ³
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	469 Tahawus Rd, Newcomb, NY 12852, USA
Wind Speed:	107 mph
Snow Load:	70 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.022069 ksf



Design Disclaimer

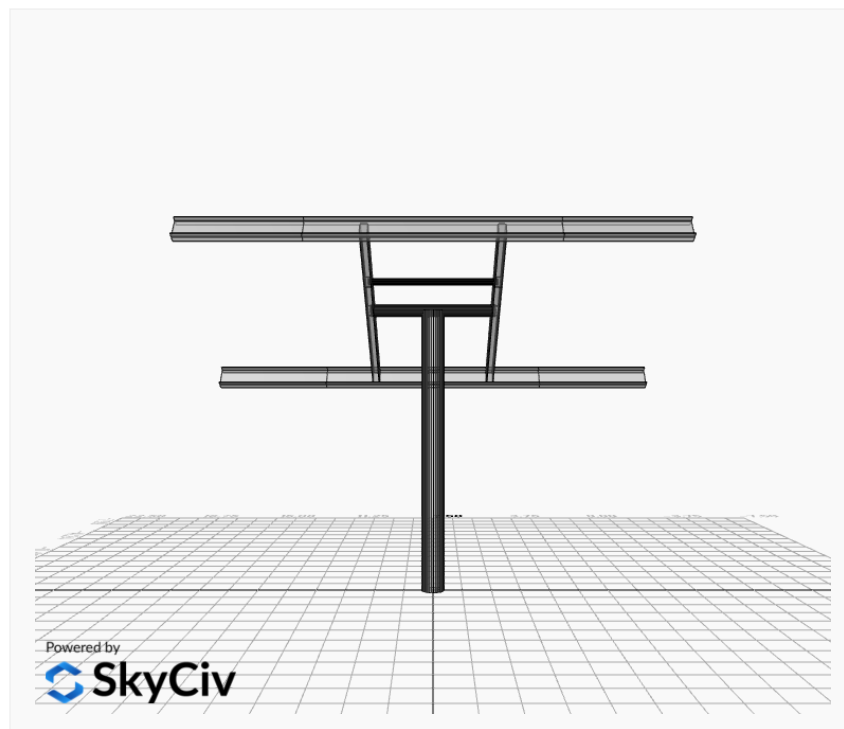
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

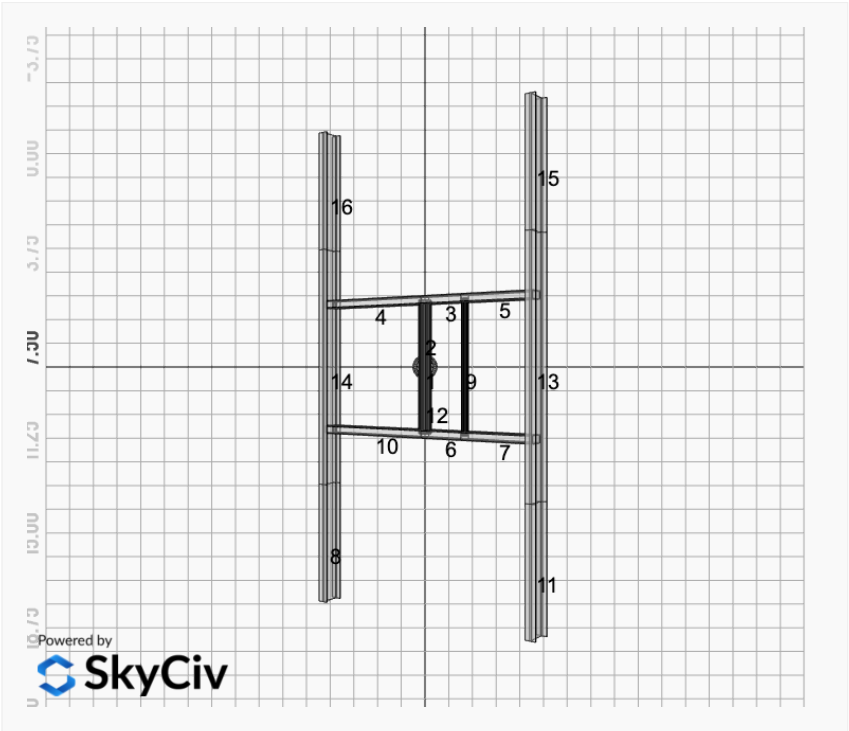
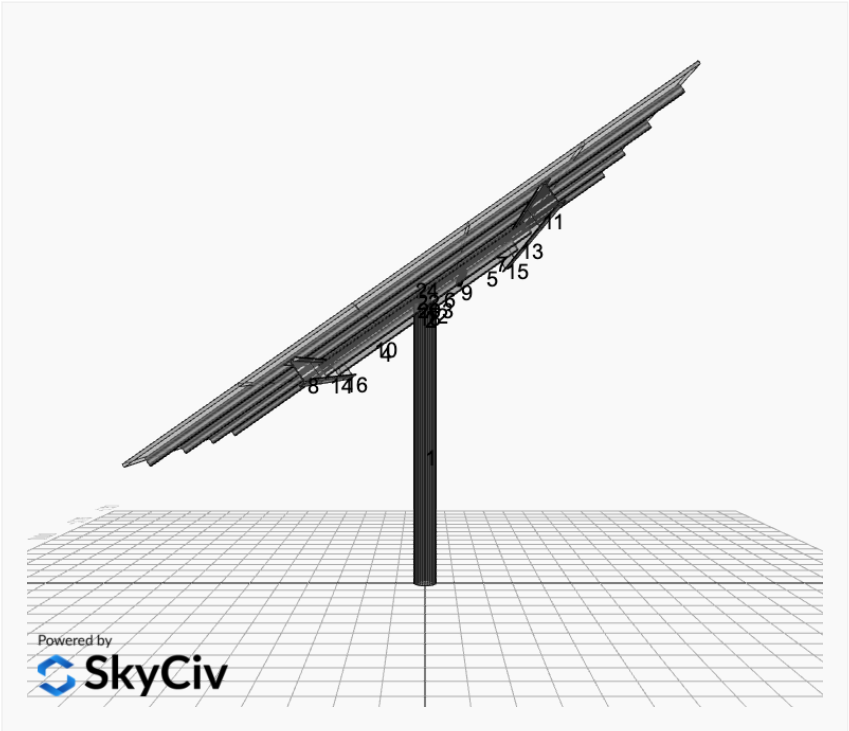
AutoDesigner Input

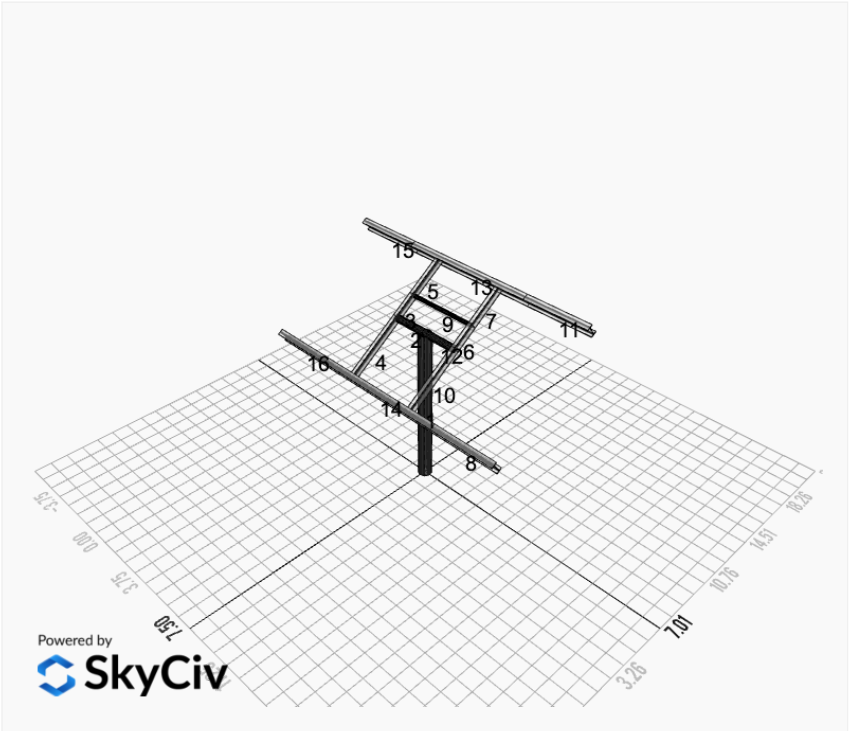
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Design Notes:

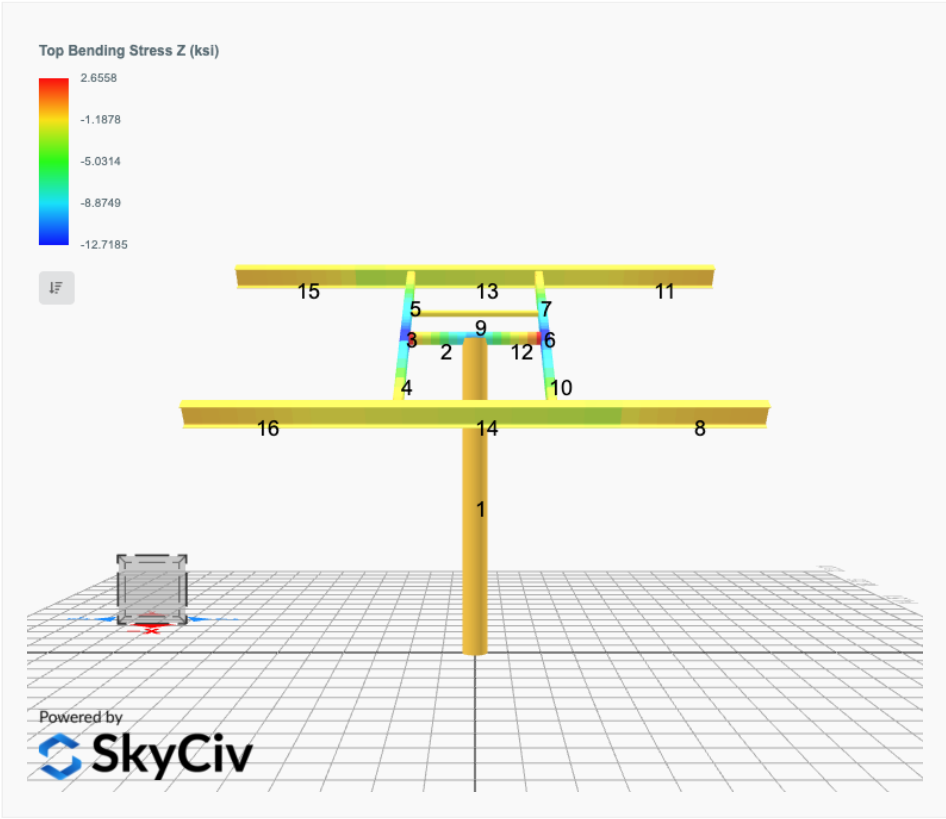
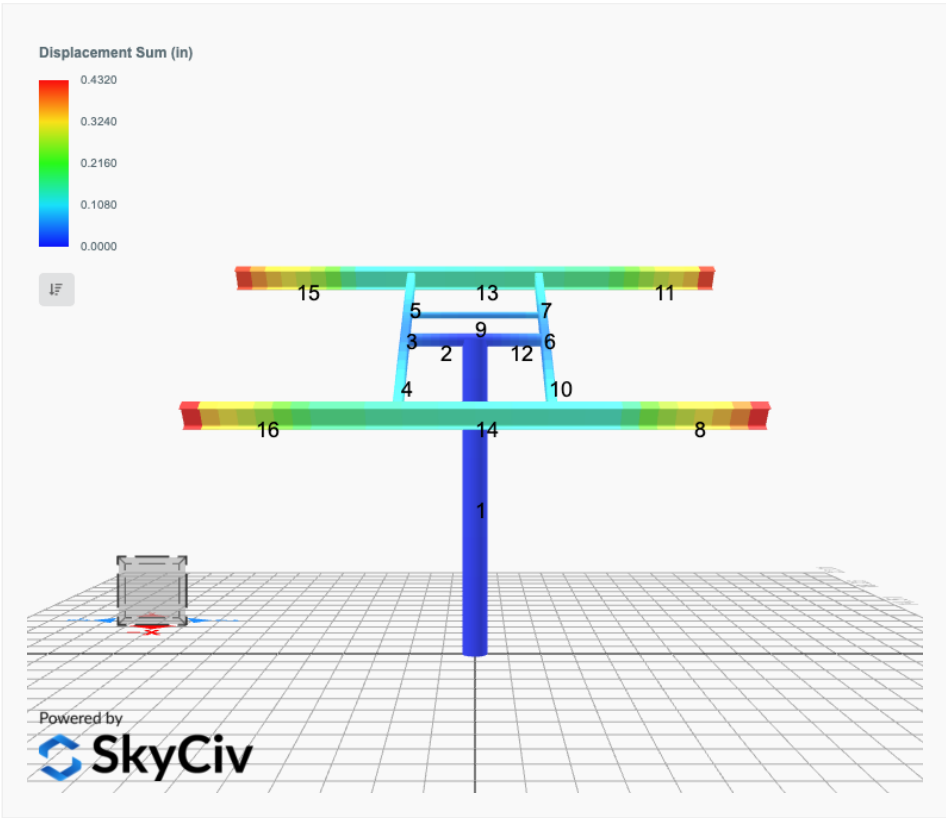
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

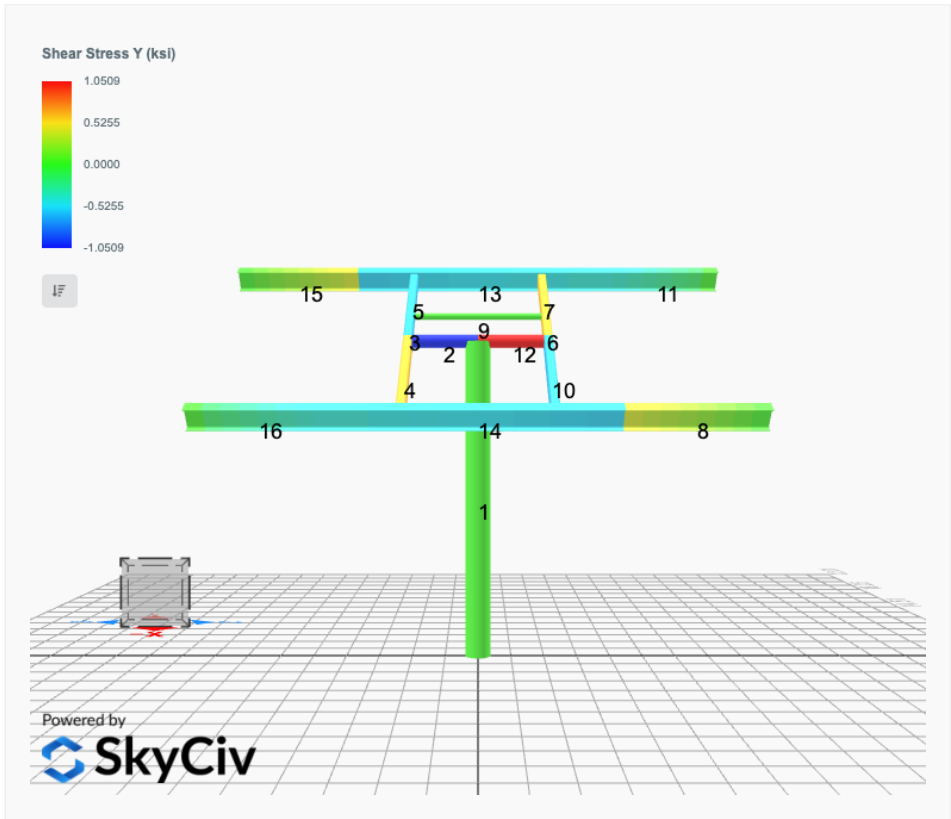
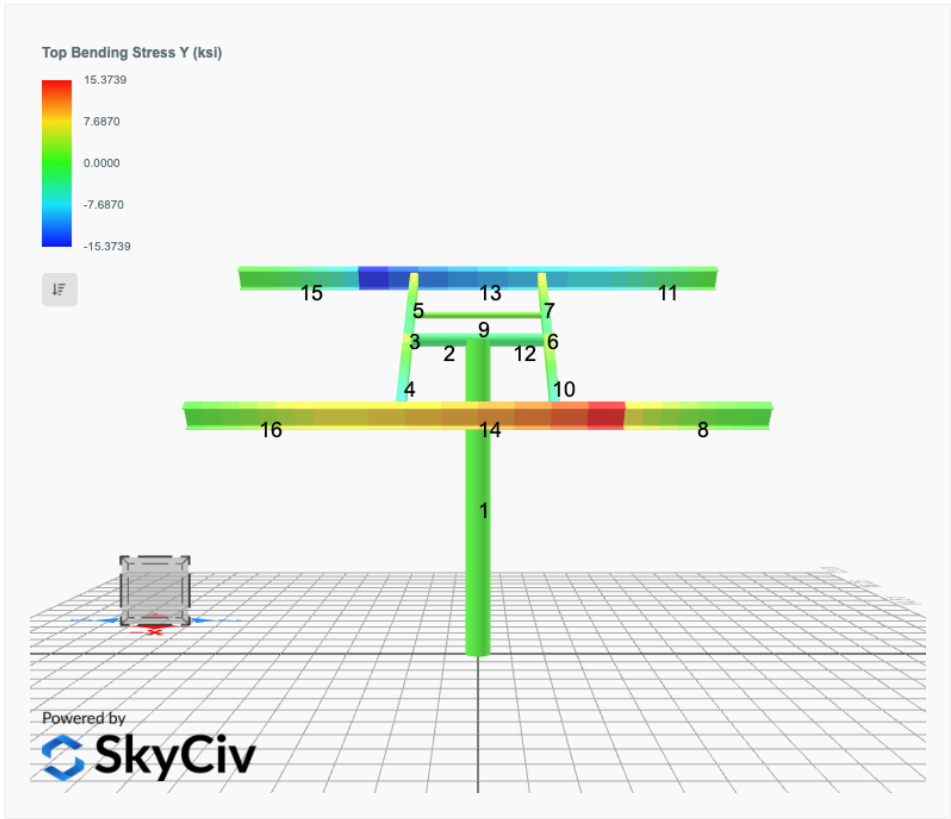


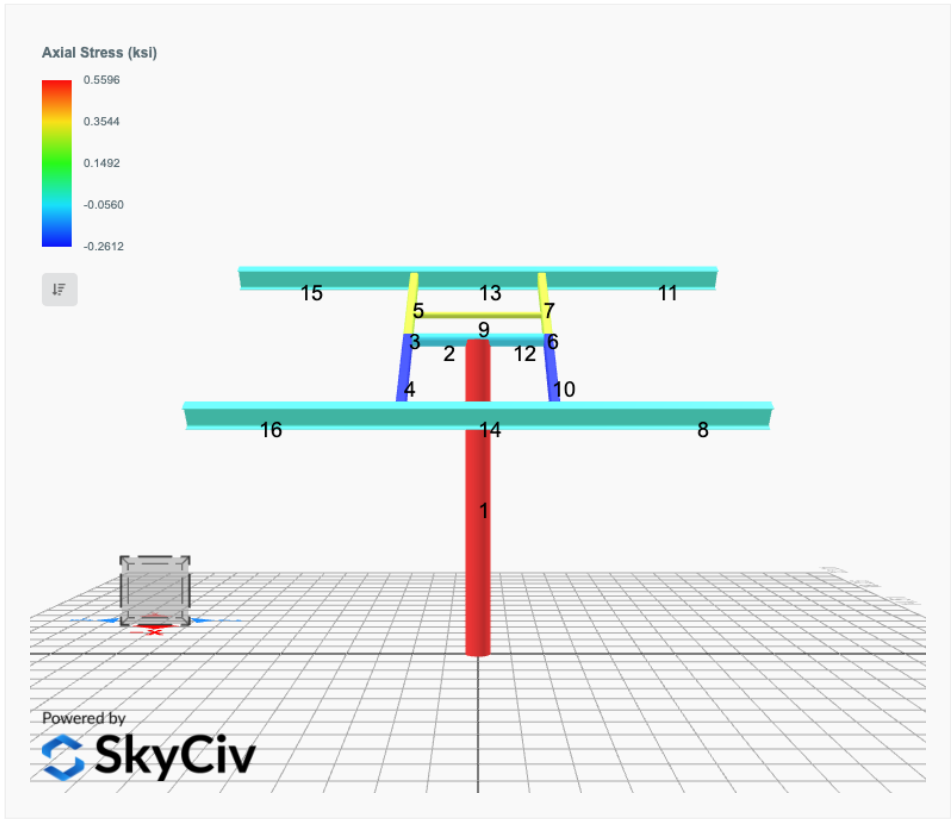




FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.9095	-0.0000	0.0000	-0.0000	0.0251
ULS: 2. D + L	-0.0000	1.9095	-0.0000	0.0000	-0.0000	0.0251
ULS: 3. D + (S or Lr or R)	-0.0000	6.6098	0.0000	0.0000	-0.0000	0.0520
ULS: 3. D + (S or Lr or R)	-0.0000	1.9095	-0.0000	0.0000	-0.0000	0.0251
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	5.4347	0.0000	0.0000	-0.0000	0.0453
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.9095	-0.0000	0.0000	-0.0000	0.0251
ULS: 5b. D + 0.7E	-0.0000	1.9095	-0.0000	0.0000	-0.0000	0.0251
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	5.4347	0.0000	0.0000	-0.0000	0.0453
ULS: 8. 0.6D + 0.7E	-0.0000	1.1457	-0.0000	0.0000	-0.0000	0.0151
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1350	4.9586	0.0000	0.0000	-0.0000	19.5801
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.1350	4.9586	0.0000	0.0000	-0.0000	19.5801
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8014	-0.6631	0.0000	0.0000	-0.0000	-16.0090
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.5012	-0.2343	0.0000	0.0000	-0.0000	-19.4011
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6012	7.7215	0.0000	0.0000	-0.0000	14.7115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6012	7.7215	0.0000	0.0000	-0.0000	14.7115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3510	3.5052	0.0000	0.0000	-0.0000	-11.9803
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1259	3.8268	0.0000	0.0000	-0.0000	-14.5244
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6012	4.1963	0.0000	0.0000	-0.0000	14.6913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6012	4.1963	0.0000	0.0000	-0.0000	14.6913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3510	-0.0199	0.0000	0.0000	-0.0000	-12.0005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1259	0.3016	0.0000	0.0000	-0.0000	-14.5446
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1350	4.1948	0.0000	0.0000	-0.0000	19.5700
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.1350	4.1948	0.0000	0.0000	-0.0000	19.5700
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8014	-1.4269	0.0000	0.0000	-0.0000	-16.0190
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5012	-0.9981	0.0000	0.0000	-0.0000	-19.4112

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
Axial	12.3527
Shear X	-3.5583
Shear Z	-0.0000
Moment X	-0.0000
Moment Z	32.9671

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
Axial	7.7215
Shear X	-2.1350
Shear Z	0.0000
Moment X	0.0000
Moment Z	19.5801

Project Details

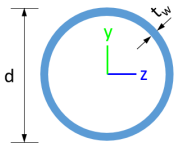
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 Provision: LRFD
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 User Name: sales@mtsolar.us
 Unit System: imperial



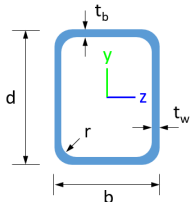
Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

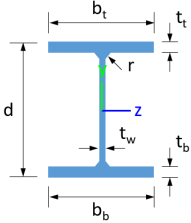
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

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					
9	8in Pipe Sch 40	8.63	0.32					

								
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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		

								
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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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

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	52.83	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	52.83	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	126.79	32.87	6.12	40.24	43.62
14	133.20	126.79	32.87	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
17	133.20	118.19	32.87	6.12	40.24	43.62
18	133.20	126.79	32.87	6.12	40.24	43.62
19	133.20	118.19	32.14	6.12	40.24	43.62
20	133.20	126.79	32.87	6.12	40.24	43.62

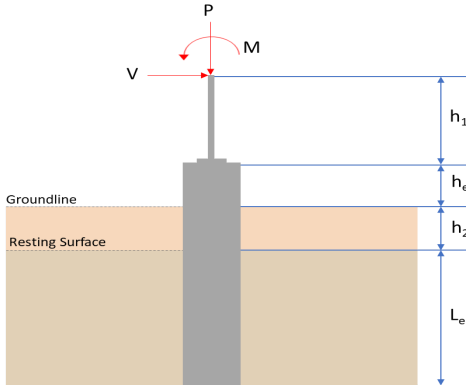
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.050	0.396	0.000	0.031	0.000	0.416	#13	0.385	Not Required	Pass
2	0.003	0.448	0.171	0.101	0.030	0.581	#21	0.035	Not Required	Pass
3	0.012	0.646	0.072	0.065	0.011	0.724	#21	0.045	Not Required	Pass
4	0.012	0.635	0.179	0.064	0.038	0.762	#21	0.080	Not Required	Pass
5	0.012	0.401	0.185	0.064	0.046	0.449	#21	0.074	Not Required	Pass
6	0.012	0.646	0.072	0.065	0.011	0.724	#21	0.045	Not Required	Pass
7	0.012	0.401	0.185	0.064	0.046	0.449	#21	0.074	Not Required	Pass
8	0.000	0.076	0.203	0.033	0.015	0.279	#21	Not Required	Not Required	Pass
9	0.014	0.052	0.050	0.001	0.000	0.109	#21	0.204	Not Required	Pass
10	0.012	0.635	0.179	0.064	0.038	0.762	#21	0.080	Not Required	Pass
11	0.000	0.076	0.203	0.033	0.015	0.280	#21	Not Required	Not Required	Pass
12	0.003	0.448	0.171	0.101	0.030	0.581	#21	0.035	Not Required	Pass
13	0.000	0.165	0.437	0.049	0.022	0.602	#21	Not Required	Not Required	Pass
14	0.000	0.163	0.437	0.048	0.022	0.600	#21	Not Required	Not Required	Pass
15	0.000	0.076	0.203	0.033	0.015	0.280	#21	Not Required	Not Required	Pass
16	0.000	0.076	0.203	0.033	0.015	0.279	#21	Not Required	Not Required	Pass
17	0.008	0.176	0.102	0.018	0.008	0.283	#21	0.124	Not Required	Pass
18	0.000	0.165	0.437	0.049	0.022	0.602	#21	Not Required	Not Required	Pass
19	0.008	0.178	0.112	0.018	0.008	0.293	#21	0.186	Not Required	Pass
20	0.000	0.163	0.437	0.048	0.022	0.600	#21	Not Required	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 _{...}	Moment of inertia about the Y axis

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																										
	<div><div>SkyCiv Foundation Design</div><div>Pile Foundation</div><div>Design Information :</div><div>Design code : IBC 2021 (International Building Code)</div><div>Unit System : Imperial</div></div>																											
	<div><div>Pile Input</div><div></div><div><div>Geometry</div><div>Pile shape: rectangular</div><div>b = 48 in - Pile width</div><div>D = 48 in - Pile depth</div><div>L = 5.75 ft - Total pile length</div><div>h1 = 0 ft - Lateral load height from the top of the pile,</div><div>h2 = 0 ft - Depth to resisting surface</div><div>he = 0 ft - Length of pile above the ground</div></div><div><div>Tabulation of Soil Parameters</div><table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table></div><div><div>Tabulation of Loads</div><table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>7.722</td><td>12.353</td></tr><tr><td>Vx (kip)</td><td>-2.135</td><td>-3.558</td></tr><tr><td>Vz (kip)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mx (kipft)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mz (kipft)</td><td>19.580</td><td>32.967</td></tr></table></div><div><div>Material Properties</div><div>f'ck = 3 ksi - Concrete strength,</div></div></div>	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.722	12.353	Vx (kip)	-2.135	-3.558	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	19.580	32.967	
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Mz (kipft)	19.580	32.967																										
	<div><div>Required depth to resist lateral loads (ASD)</div><div>H - Point of application of the lateral load</div><div><div><div><div>$H = h_1 + h_2 + h_e$</div><div>$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$</div><div>$H = 0 \text{ ft}$</div></div></div><div><div>Considering x-direction:</div><div>Ho - Lateral force per length of pile,</div><div><div><div>$H_o = \frac{V_x}{1.57 D}$</div><div>$H_o = \frac{(-2.135 \text{ kip})}{1.57 \times (48 \text{ in})}$</div><div>$H_o = -0.33997 \text{ kip/ft}$</div></div></div><div><div>Mo - Moment per length of pile,</div><div><div>$M_o = \frac{M_z + (V_x H)}{1.57 D}$</div></div></div></div></div></div>																											

	$M_o = \frac{(19.58 \text{ kipft}) + ((-2.135 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.1178 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation:</p> $L_{e,x} = 5.227 \text{ ft} - \text{Required depth in x-direction,}$ <p>Considering z-direction:</p> $L_{e,z} = 0 \text{ ft} - \text{Required depth in z-direction,}$ <p>Minimum embedded depth required:</p> $L_{e,req} - \text{Depth of pile required,}$ $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.227 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.227 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.227 \text{ ft})}{(5.75 \text{ ft})}$ $\text{Ratio} = 0.90904$	<p>Status: PASS Ratio: 0.910</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(7.722 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.48262 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.48262 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.24131$	<p>Status: PASS Ratio: 0.240</p>
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.33997$ kip/ft - Lateral force per length of pile,

$M_o = 3.1178$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1178 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.33997 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.1178 \text{ kipft/ft})) + (4 \times (-0.33997 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9746 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1178 \text{ kipft/ft})) + (3 \times (-0.33997 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^3 \times [(3 \times (3.1178 \text{ kipft/ft})) + (2 \times (-0.33997 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.18189 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.1178 \text{ kipft/ft})) + ((-0.33997 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.77687 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9746 \text{ ft})}{2}$$

$$p_a = 0.29809 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18189 \text{ kip/ft}^2)}{(0.29809 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.61018$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

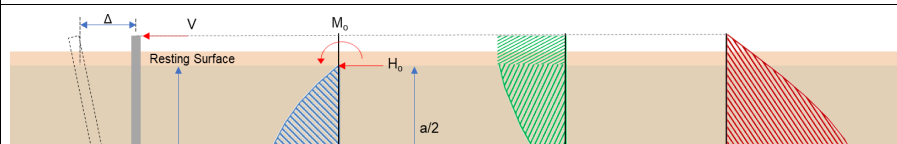
$$\text{Ratio} = \frac{s}{p_s}$$

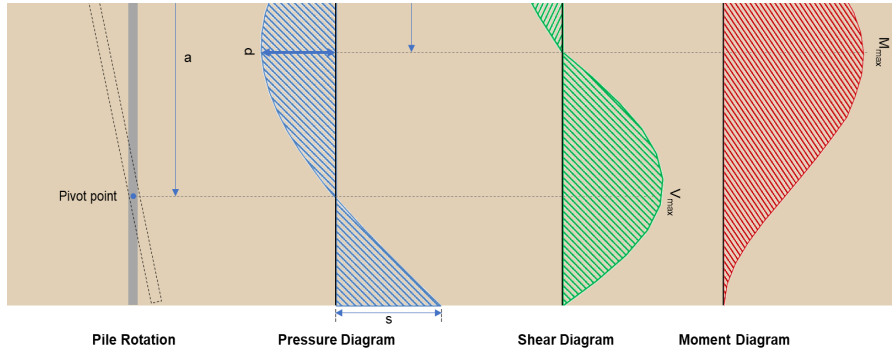
$$\text{Ratio} = \frac{(0.77687 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90071$$

Status: **PASS**
Ratio: **0.610**

Status: **PASS**
Ratio: **0.900**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.558 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.56656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.967 \text{ kipft}) + ((-3.558 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.2495 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.2495 \text{ kipft/ft})}{(-0.56656 \text{ kip/ft})}$$

$$E = 9.2656 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.2495 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.56656 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.2495 \text{ kipft/ft})) + (4 \times (-0.56656 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9736 \text{ ft}$$

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

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

$$V_{max} = ((-0.56656 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.2656 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9736 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.2656 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9736 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.9563 \text{ kip}$$

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

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

$$M_{max} = ((-0.56656 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(9.2656 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9736 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.2656 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9736 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.2656 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9736 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 21.692 \text{ kipft}$	
</			

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 3183.4 \text{ kip}$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(12.353 \text{ kip})}{(3183.4 \text{ kip})}$ $\text{Ratio} = 0.0038804$	Status: PASS Ratio: 0.000
22.5.2.2	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	<p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 324.49 \text{ kip}$	
22.5.5.1.1(a)	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 12.353 \text{ kip} \rightarrow 12353 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(12353 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 131.44 \text{ kip}$	
22.5.5.1.2	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b)</p> $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 406.27 \text{ kip}$ <p>V_c - Governing shear strength of concrete</p> $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(324.49 \text{ kip}), (131.44 \text{ kip}), (406.27 \text{ kip})]$ $V_c = 131.44 \text{ kip}$	

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.44 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.52 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.9563 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(7.9563 \text{ kip})}{(118.52 \text{ kip})}$ $Ratio = 0.067132$	<p>Status: PASS Ratio: 0.070</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kip ft}$ <p>$\phi M_{n,2}$</p>	

	$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2545.9 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$ $\phi M_n = 273.42 \text{ kipft}$ <p>Considering x-direction:</p> <p>$M_{max} = 21.692 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(21.692 \text{ kipft})}{(273.42 \text{ kipft})}$ $\text{Ratio} = 0.079337$	
		Status: PASS Ratio: 0.080