

Your Project Calculations



Project Name: MTSOLAR_4G86356GGHDDD

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_4G86356GGHDDD&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=6JeDydMGnIAjNtmuMxF37NwUeINHITFvalT7dYCeUBHIZIzBYtOSh5I6QCxbpNK8

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-57-L-4Hx3W-A42A
Duty Classification:	SD
Module Width:	41.14 in
Module Length:	67.60in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	15.57 ft
Total Frame Length:	17.00 ft
Frame Weight:	753 lbs
Array Dimensions N/S:	13.88 ft
Array Dimensions E/W:	17.15 ft
Rail Length:	166.57 in
Rail Spacing:	2.82 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	10.32 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 9.00 ft
Foundation Volume:	2.356 y ³
Foundation Result:	PASSED
Mount Twist:	0.000006 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	7265 W Glenmere Rd, Empire, MI 49630, USA
Wind Speed:	115 mph
Snow Load:	60 psf
Design Uplift Pressure:	0.025363 ksf
Design Downforce Pressure:	-0.025363 ksf
Design Snow Pressure:	0.013196 ksf



Design Disclaimer

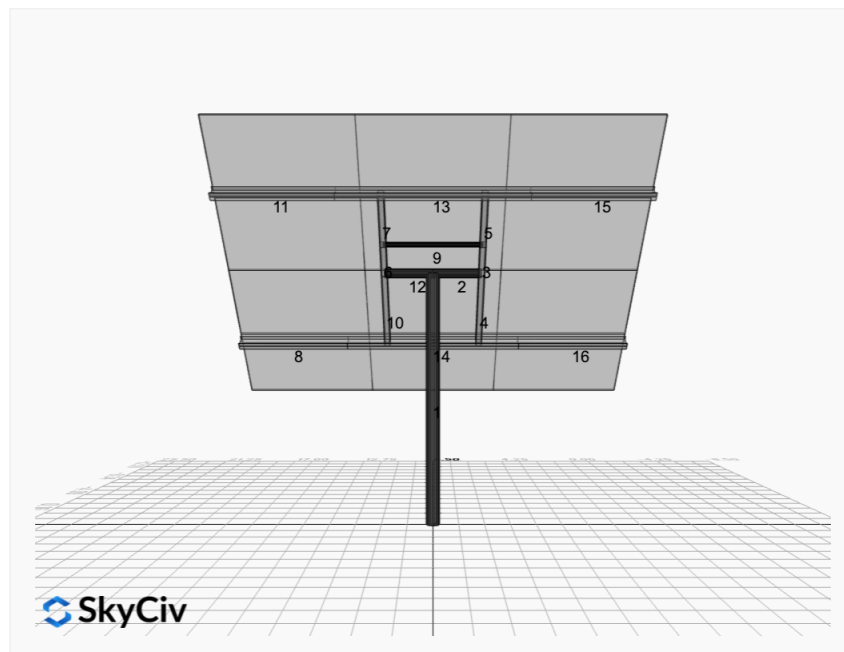
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

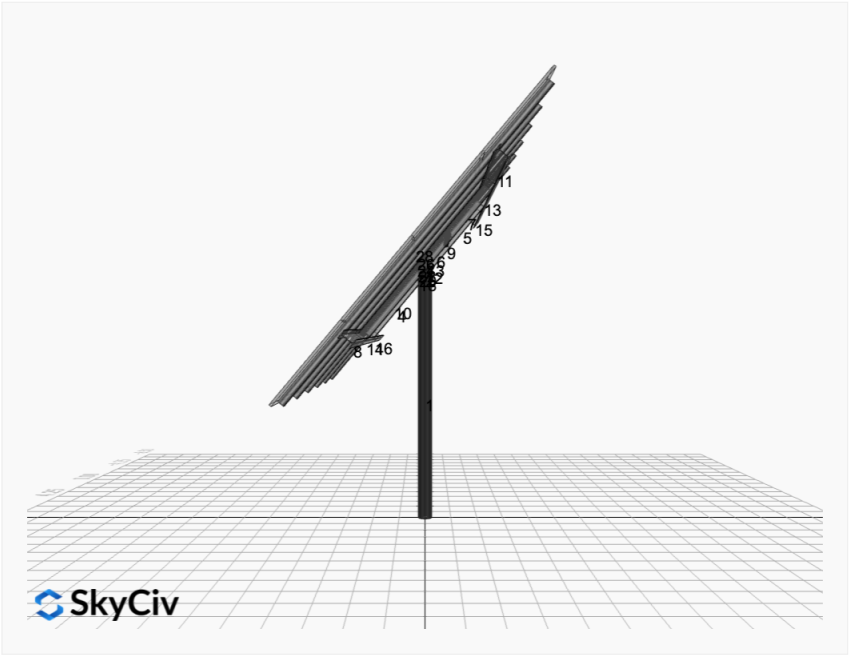
AutoDesigner Input

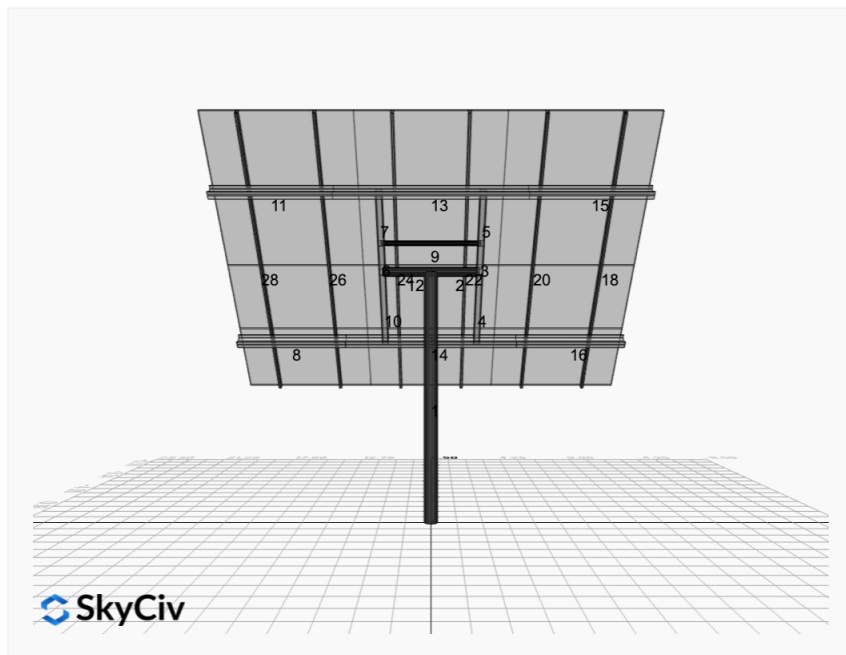
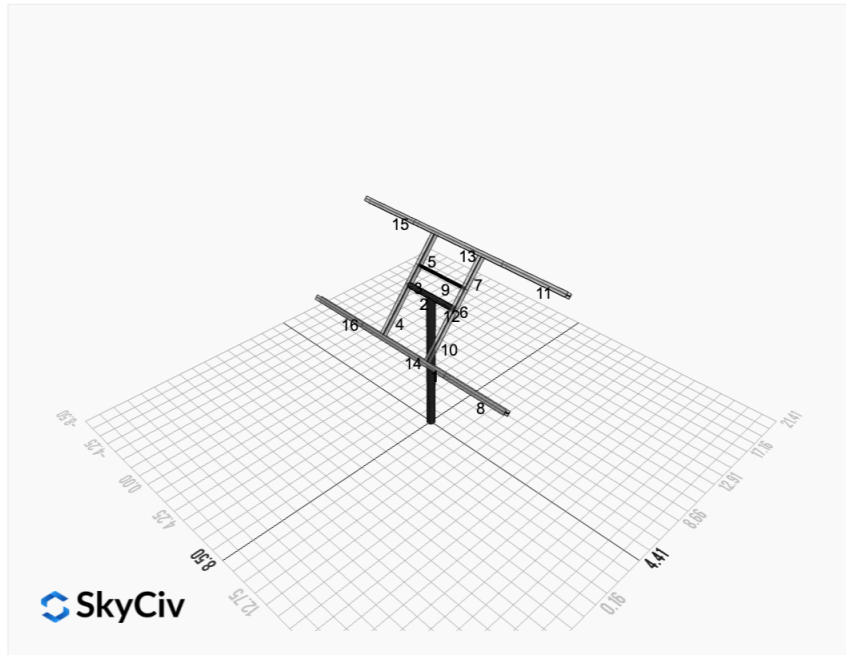
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Design Notes:

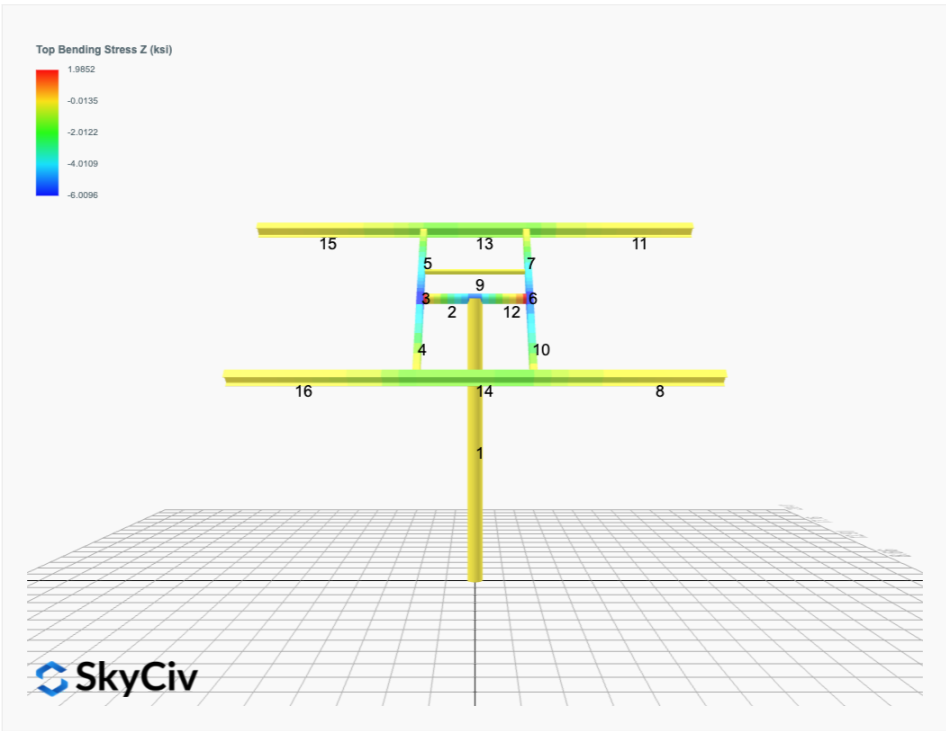
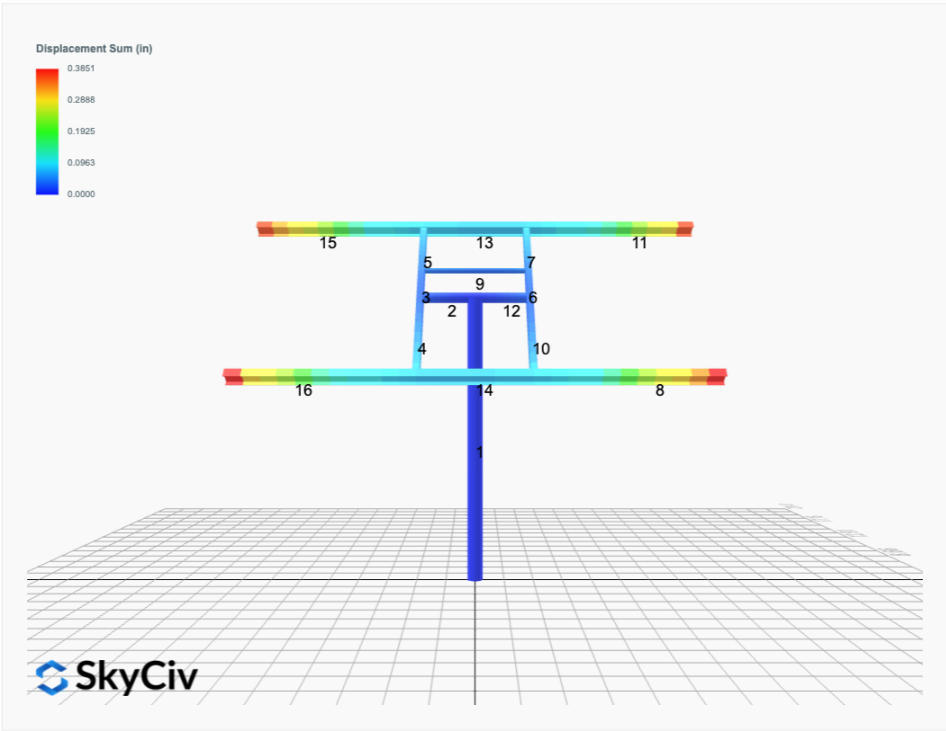
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

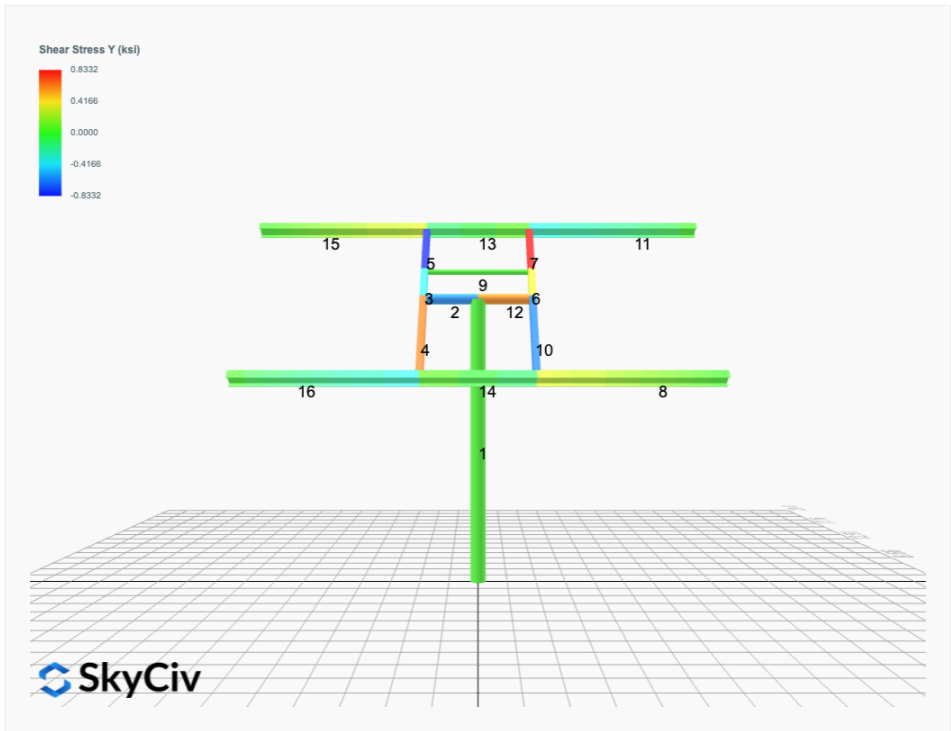
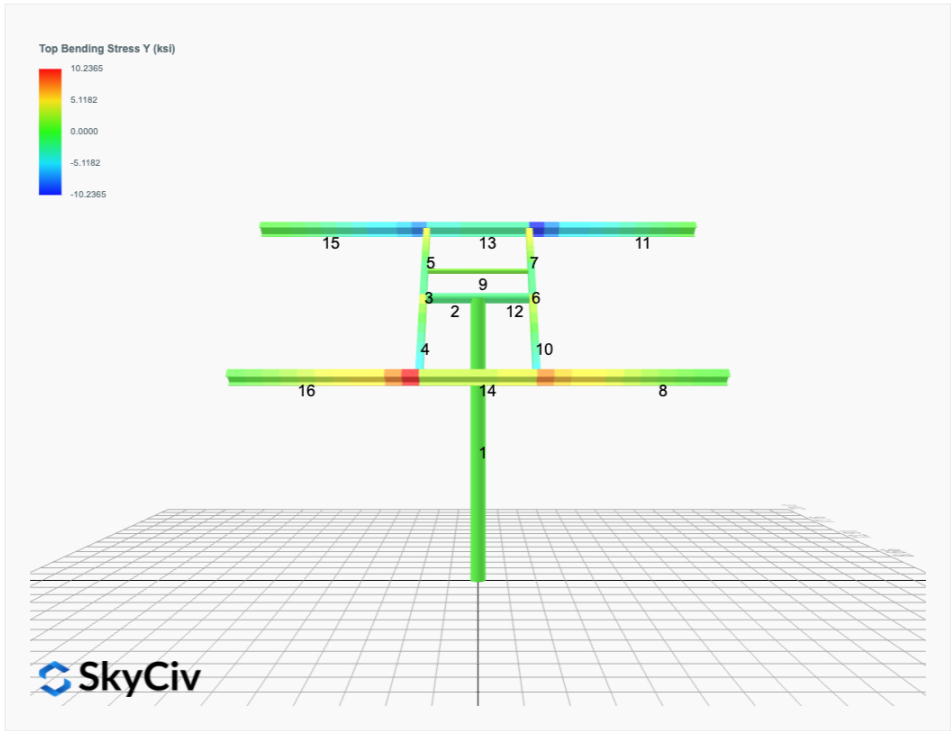


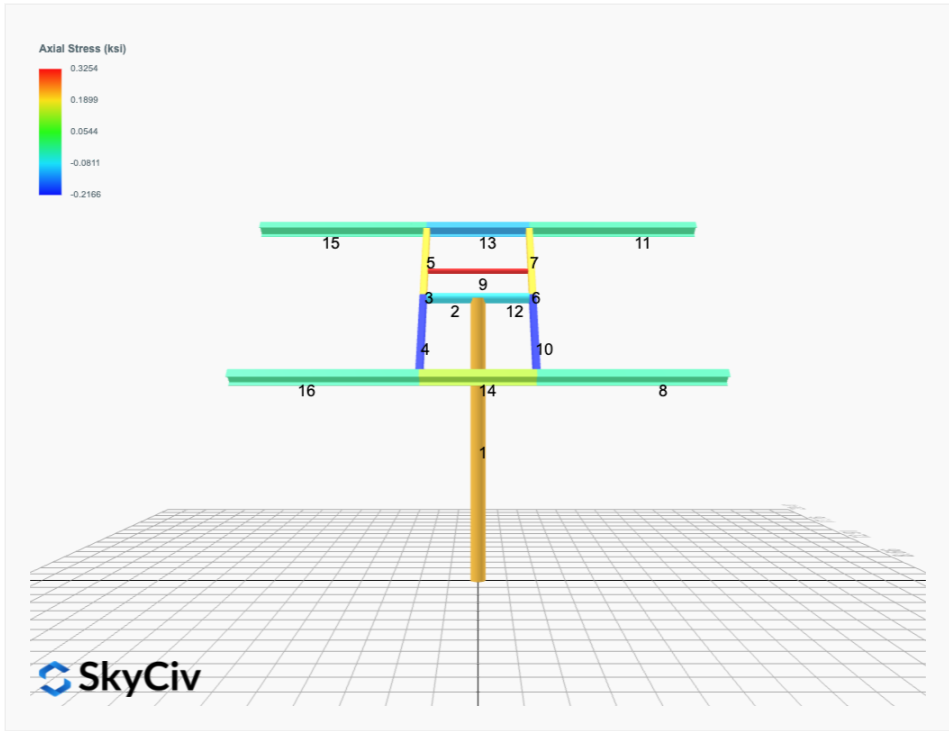




FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 2. D + L	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 3. D + (S or Lr or R)	0.0000	3.8870	0.0000	0.0000	-0.0000	0.0231
ULS: 3. D + (S or Lr or R)	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.3866	0.0000	0.0000	-0.0000	0.0213
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 5b. D + 0.7E	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.3866	0.0000	0.0000	-0.0000	0.0213
ULS: 8. 0.6D + 0.7E	0.0000	1.1313	0.0000	0.0000	-0.0000	0.0095
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7751	4.2140	0.0000	0.0000	-0.0000	29.1385
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7751	-0.4431	0.0000	0.0000	-0.0000	-28.1395
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0813	5.1330	0.0000	0.0000	-0.0000	21.8633
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.3866	0.0000	0.0000	-0.0000	0.0213
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0813	1.6402	0.0000	0.0000	-0.0000	-21.0952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.3866	0.0000	0.0000	-0.0000	0.0213
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0813	3.6319	0.0000	0.0000	-0.0000	21.8578
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0813	0.1391	0.0000	0.0000	-0.0000	-21.1006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.8855	0.0000	0.0000	-0.0000	0.0159
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7751	3.4598	0.0000	0.0000	-0.0000	29.1321
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.1313	0.0000	0.0000	-0.0000	0.0095
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7751	-1.1973	0.0000	0.0000	-0.0000	-28.1459
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.1313	0.0000	0.0000	-0.0000	0.0095

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	7.4054
Shear X	-4.6251
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	49.3371

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	5.1330
Shear X	-2.7751
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	29.1385

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States

User Name: sales@mtsolar.us
 Project Name: MTSOLAR_4G86356GGHDDD
 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)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
8	6in Pipe Sch 80	6.63	0.43					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31

8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D	
1	8	21.66	21.66	10.32	-	300	200	1	
2	4	1.30	1.30	2.00	-	300	200	1	
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.24,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.81,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.76,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1	
6	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.24,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1	
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.76,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1	
8	18	9.97	9.97	4.75	2.33,2.33	300	200	1	
9	1	2.60	2.60	4.00	-	300	200	1	
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.81,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1	
11	18	9.97	9.97	4.75	2.33,2.33	300	200	1	
12	4	1.30	1.30	2.00	-	300	200	1	
13	18	4.88	4.00	7.50	1.03,1.03	300	200	1	
14	18	4.88	4.00	7.50	1.03,1.03	300	200	1	
15	18	9.97	9.97	4.75	2.33,2.33	300	200	1	
16	18	9.97	9.97	4.75	2.33,2.33	300	200	1	

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	378.22	135.34	62.23	62.23	113.47	113.47
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	34.69	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51

10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	34.69	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.03	18.15	6.45	30.09	45.74
14	120.60	84.03	18.13	6.45	30.09	45.74
15	120.60	34.69	23.36	6.45	30.09	45.74
16	120.60	34.69	23.36	6.45	30.09	45.74

Design Ratio

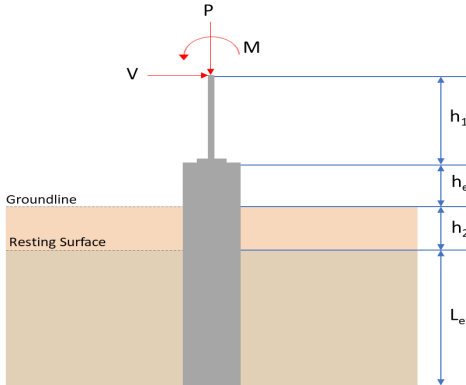
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.055	0.793	0.000	0.041	0.000	0.819	#13	0.592	Not Required	Pass
2	0.004	0.353	0.293	0.082	0.054	0.647	#13	0.034	Not Required	Pass
3	0.013	0.676	0.108	0.068	0.005	0.742	#13	0.044	Not Required	Pass
4	0.012	0.672	0.357	0.067	0.043	0.804	#13	0.078	Not Required	Pass
5	0.013	0.420	0.374	0.067	0.055	0.473	#13	0.073	Not Required	Pass
6	0.013	0.676	0.108	0.068	0.005	0.742	#13	0.044	Not Required	Pass
7	0.013	0.420	0.374	0.067	0.055	0.473	#13	0.073	Not Required	Pass
8	0.000	0.111	0.194	0.036	0.012	0.282	#21	Not Required	Not Required	Pass
9	0.019	0.049	0.068	0.001	0.000	0.121	#13	0.198	Not Required	Pass
10	0.012	0.672	0.357	0.067	0.043	0.804	#13	0.078	Not Required	Pass
11	0.000	0.111	0.194	0.036	0.012	0.282	#21	Not Required	Not Required	Pass
12	0.004	0.353	0.293	0.082	0.054	0.647	#13	0.034	Not Required	Pass
13	0.008	0.274	0.364	0.050	0.016	0.576	#21	0.177	Not Required	Pass
14	0.009	0.279	0.364	0.050	0.016	0.576	#21	0.177	Not Required	Pass
15	0.000	0.111	0.194	0.036	0.012	0.282	#21	Not Required	Not Required	Pass
16	0.000	0.111	0.194	0.036	0.012	0.282	#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 _{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

ON
NG

Capacity is provided
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: round</div><div>D = 36 in - Pile diameter</div><div>Le = 9 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>5.133</td><td>7.405</td></tr><tr><td>Vx (kip)</td><td>-2.775</td><td>-4.625</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>29.138</td><td>49.337</td></tr></table></div><div><div>Material Properties</div><div>f'ck = 2.5 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)	5.133	7.405	Vx (kip)	-2.775	-4.625	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	29.138	49.337	
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																									
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Mx (kipft)	0.000	0.000																										
Mz (kipft)	29.138	49.337																										
	<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>$H_o = \frac{V_x}{D}$</div><div>$H_o = \frac{(-2.775 \text{ kip})}{(36 \text{ in})}$</div><div>$H_o = -0.925 \text{ kip/ft}$</div></div></div><div><div>Mo - Moment per length of pile,</div><div><div>$M_o = \frac{M_z + (V_x H)}{D}$</div></div></div></div></div></div>																											

	$M_o = \frac{(29.138 \text{ kipft}) + ((-2.775 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 9.7127 \text{ kipft/ft}$ <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: $L_{e,x} = 8.0392 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(8.0392 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 8.039 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 9 \text{ ft}$ <p>Ratio - Embedded depth</p> $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(8.039 \text{ ft})}{(9 \text{ ft})}$ $Ratio = 0.89322$	Status: PASS Ratio: 0.890
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$ $A = 7.0686 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_o}{A}$ $q = \frac{(5.133 \text{ kip})}{(7.0686 \text{ ft}^2)}$ $q = 0.72617 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p>Ratio - Capacity</p> $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.72617 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.36309$	Status: PASS Ratio: 0.360
Czerniak	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(9 \text{ ft})}{(36 \text{ in})}$	

$$L/D = 3$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.925$ kip/ft - Lateral force per length of pile,

$M_o = 9.7127$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7127 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.925 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (9.7127 \text{ kipft/ft})) + (4 \times (-0.925 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2727 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (9.7127 \text{ kipft/ft})) + (3 \times (-0.925 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^3 \times [(3 \times (9.7127 \text{ kipft/ft})) + (2 \times (-0.925 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.22422 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (9.7127 \text{ kipft/ft})) + ((-0.925 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2916 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2727 \text{ ft})}{2}$$

$$p_a = 0.47045 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22422 \text{ kip/ft}^2)}{(0.47045 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.4766$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (9 \text{ ft})$$

$$p_s = 1.35 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

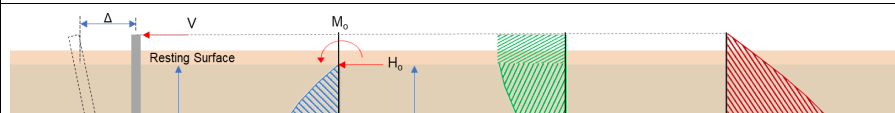
$$\text{Ratio} = \frac{s}{p_s}$$

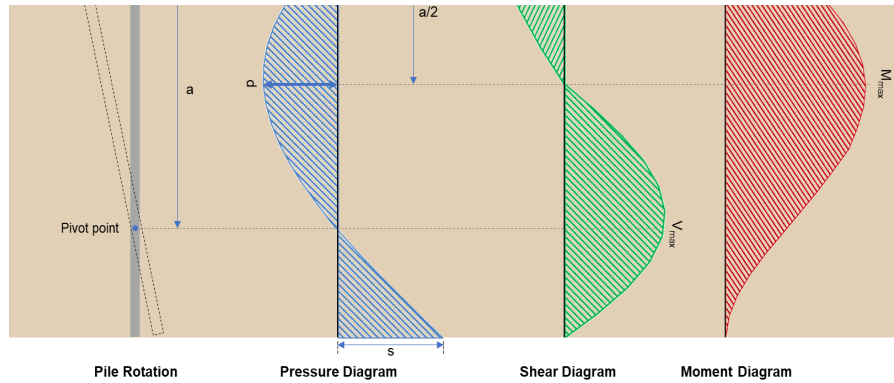
$$\text{Ratio} = \frac{(1.2916 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95675$$

Status: **PASS**
Ratio: **0.480**

Status: **PASS**
Ratio: **0.960**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.625 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.5417 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.337 \text{ kipft}) + ((-4.625 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 16.446 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.446 \text{ kipft/ft})}{(-1.5417 \text{ kip/ft})}$$

$$E = 10.667 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.446 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.5417 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (16.446 \text{ kipft/ft})) + (4 \times (-1.5417 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.27 \text{ ft}$$

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

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

$$V_{max} = ((-1.5417 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.667 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.27 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.667 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.27 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.752 \text{ kip}$$

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

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

$$M_{max} = ((-1.5417 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(10.667 \text{ ft})}{(9 \text{ ft})} + \frac{(6.27 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.667 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.27 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.667 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.27 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 53.622 \text{ kipft}$	
Table 22.4.2.1	22.4.2.2, 10.6.1.1	<p>Minimum Reinforcement Check (LRFD)</p> <p>Parameters:</p> <p>$f'_{ck} = 2.5 \text{ ksi}$ - 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,</p> <p>Longitudinal reinforcement:</p> <p>Required reinforcement due to axial load, $A_{st,required}$</p> $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(7.405 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.142 \text{ in}^2$ <p>A_{min} - Governing minimum reinforcement area,</p> $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.142 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ <p>n_{rebar} - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 6$ <p>A_{st} - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 1.8408 \text{ in}^2$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $\text{Ratio} = 0.99533$ <p>25.2.3 s_{rebar} - Minimum spacing of reinforcement,</p> $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\emptyset$: Use #3(0.375 in)</p> <p>25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,</p> $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p>	<p>Status: PASS Ratio: 1.000</p>

	<p>Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	
22.4.2.2	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.85 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$ $\phi P_N = 1253.9 \text{ kip}$ <p>Ratio - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.405 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0059055$	<p>Status: PASS Ratio: 0.010</p>
22.5.2.2	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$	
22.5.5.1.3	<p>λ_s - size effect modification factor</p> $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$	
22.5.5.1.1	<p>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</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$	
22.5.5.1.1(a)	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.405 \text{ kip} \rightarrow 7405 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(7405 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 75.695 \text{ kip}$	
22.5.5.1.2	<p>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)</p> $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,b} = 204.04 \text{ kip}$ <p>V_c - Governing shear strength of concrete</p> $V_c = Min [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = Min [(186.09 \text{ kip}), (75.695 \text{ kip}), (204.04 \text{ kip})]$	

<p>22.5.1.2</p>	<p style="text-align: center;">$V_c = 75.695 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ <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> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((75.695 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.012 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 12.752 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(12.752 \text{ kip})}{(74.012 \text{ kip})}$ $Ratio = 0.17229$ <p style="text-align: right;">Status: PASS Ratio: 0.170</p>	
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4580.4 \text{ in}^3$ <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> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ <p>$\phi M_{n,2}$</p>	

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

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

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

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:

$M_{max} = 53.622 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(53.622 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.8645$$

Status: **PASS**
Ratio: **0.860**