

Project Details



Project Name: Youngla

Date: Mon Feb 17 2025

Location: 200 West Rd, Ellington, CT 06029, USA

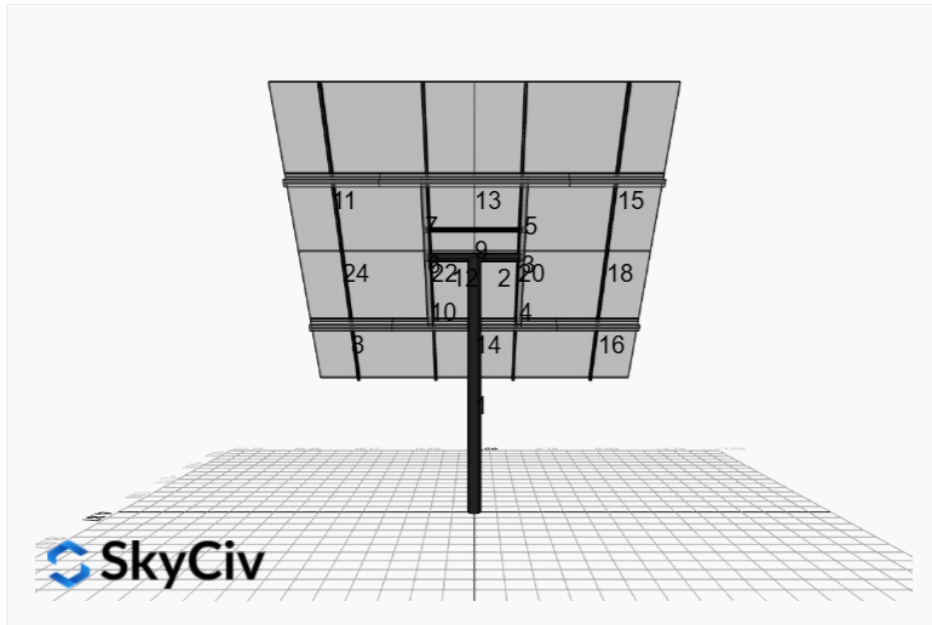
Number of Modules: 8

Unique ID: 1P-0-6TOP-SD-45-L-4Hx2W-AHF1

Number of Poles: 1

Dealer: _____

Date Sold: _____



Array Dimensions N/S	15.17 ft
Array Dimensions E/W	15.17 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

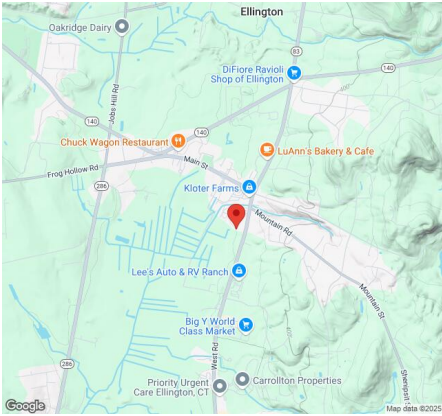
MT Solar Bill of Materials (1P-0-6TOP-SD-45-L-4Hx2W-AHF1)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-45	45IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	2

Rail Bill of Materials

Part	Qty
Rails (180in)	4
Rail Attachment	8
Module Mid Clamp	12
Module End Clamp	8
Ground Lug	2

Site Details:



Site Address: 200 West Rd, Ellington, CT 06029, USA

Array Specification

Duty Classification:	SD
Module Width:	45.00 in
Module Length:	90.00in
Number of Rows:	4
Number of Columns:	2
Total Number of Modules:	8
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	16.62 ft
Total Frame Length:	15.00 ft
Frame Weight:	902 lbs
Array Dimensions N/S:	15.17 ft
Array Dimensions E/W:	15.17 ft
Rail Length:	182.00 in
Rail Spacing:	3.79 ft

Support Specifications

Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	10.81 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

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	200 West Rd, Ellington, CT 06029, USA
Wind Speed:	109 mph
Snow Load:	35 psf

Design Disclaimer

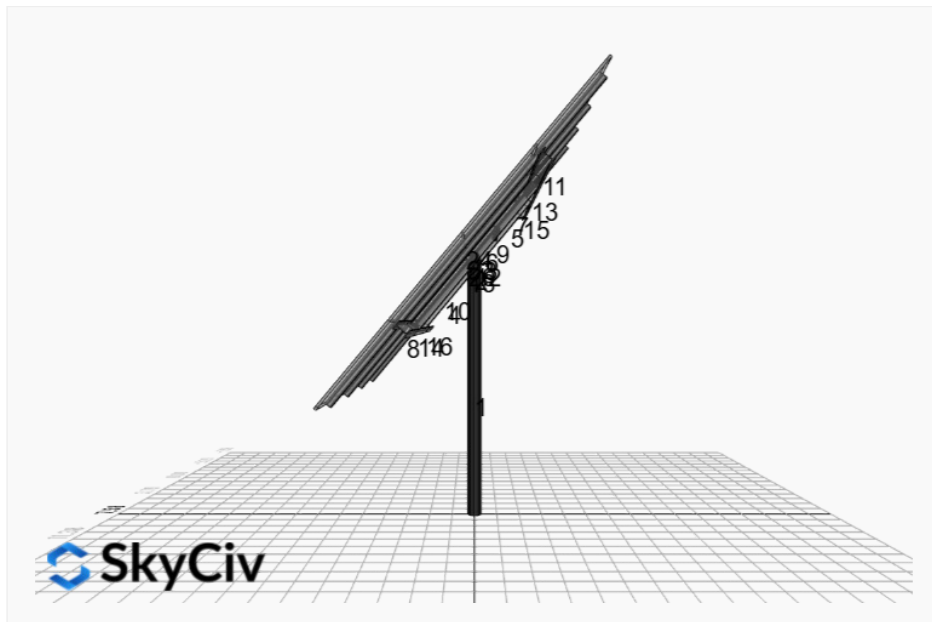
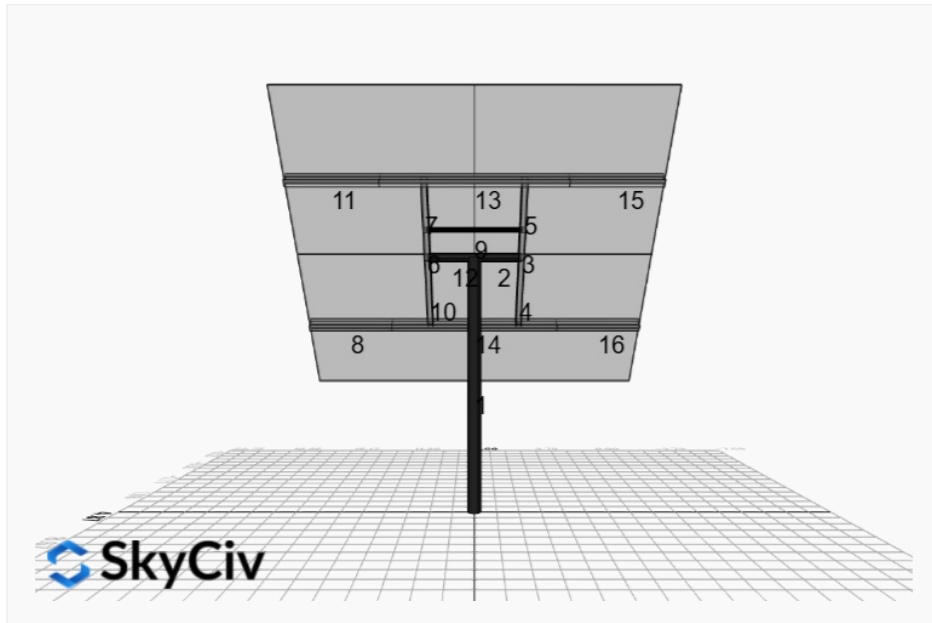
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

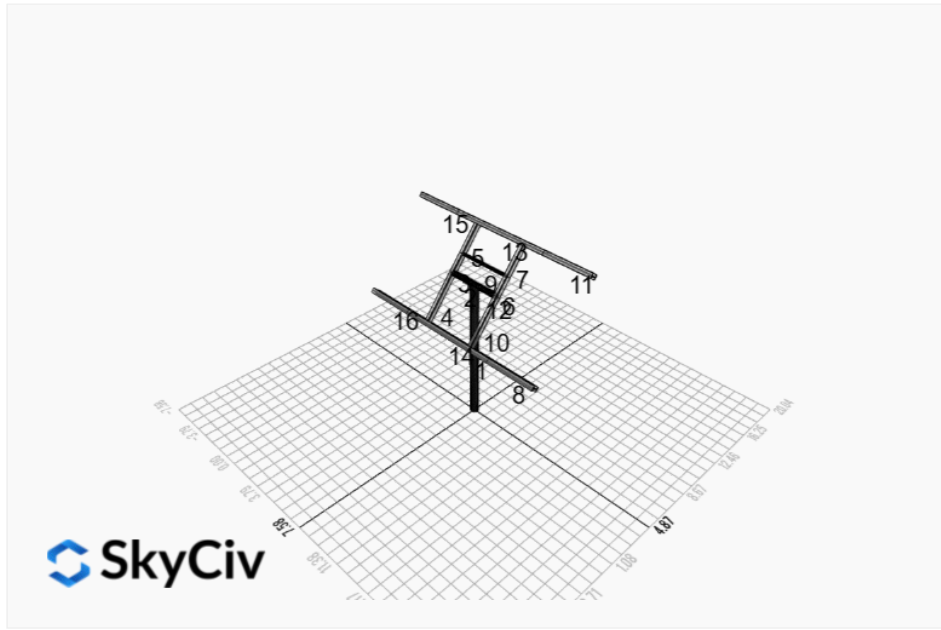
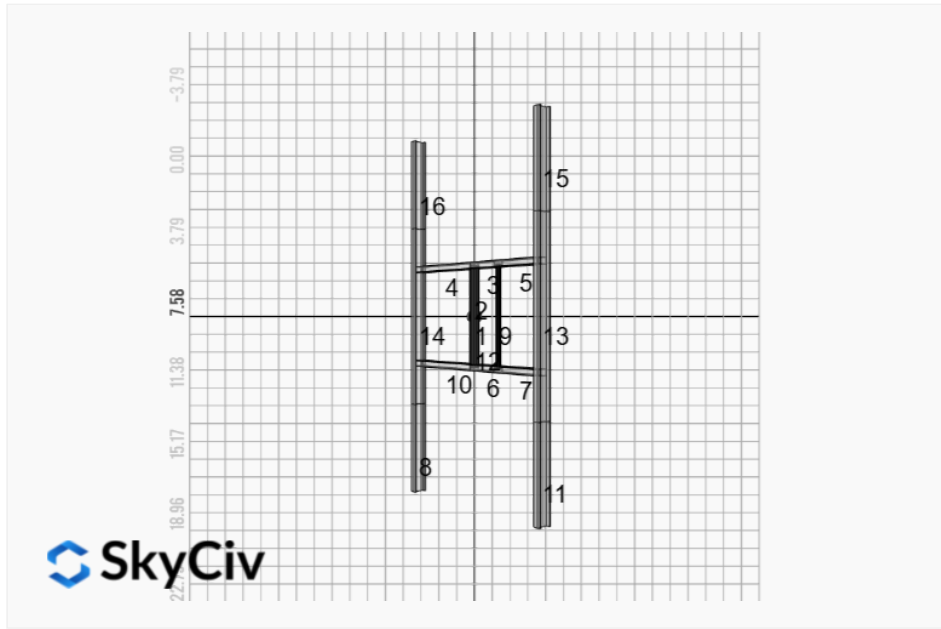
AutoDesigner Input

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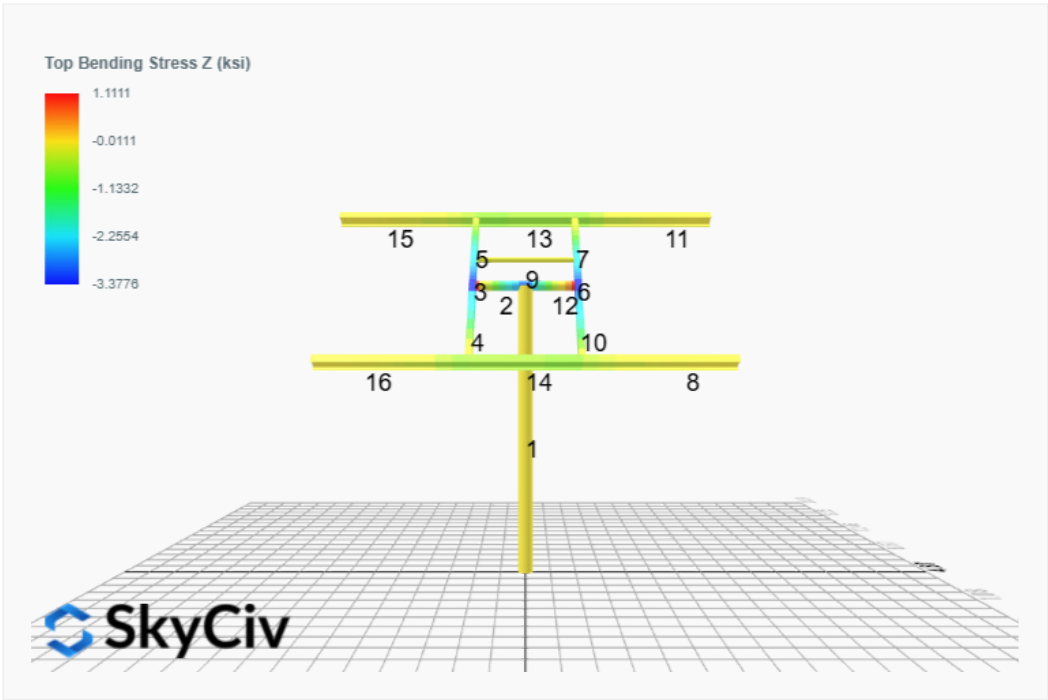
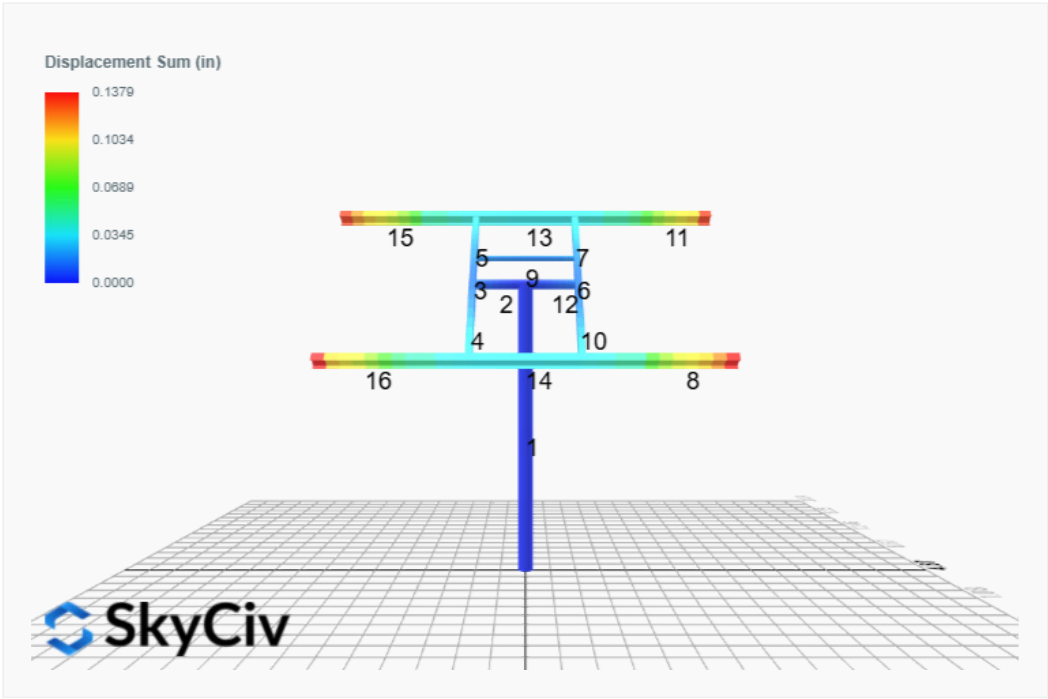
Design Notes:

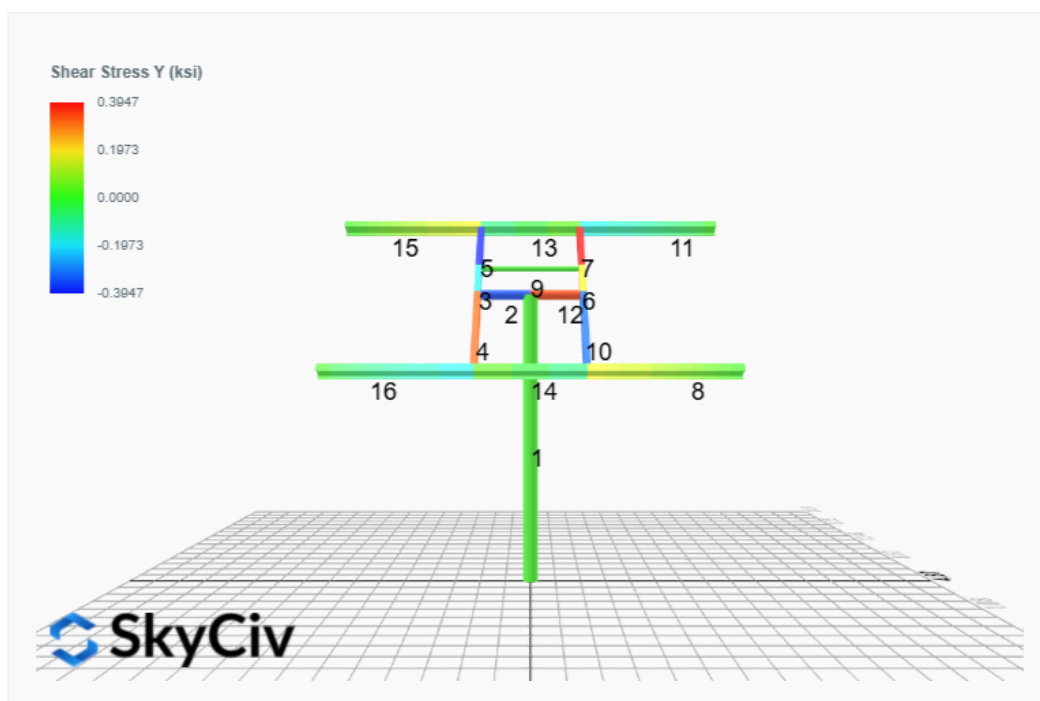
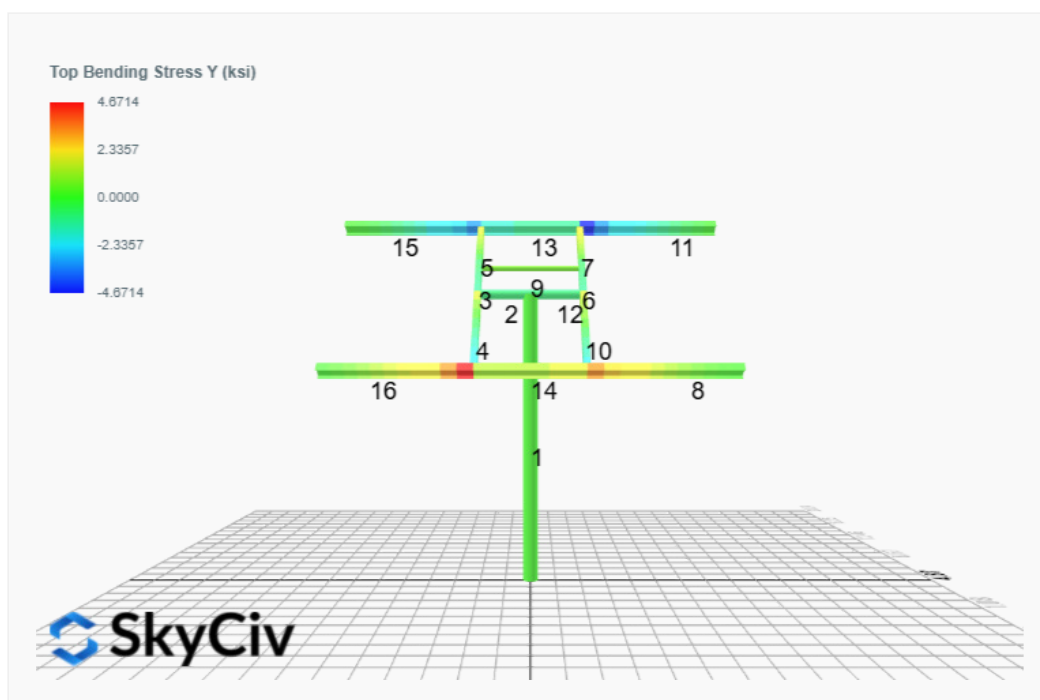
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

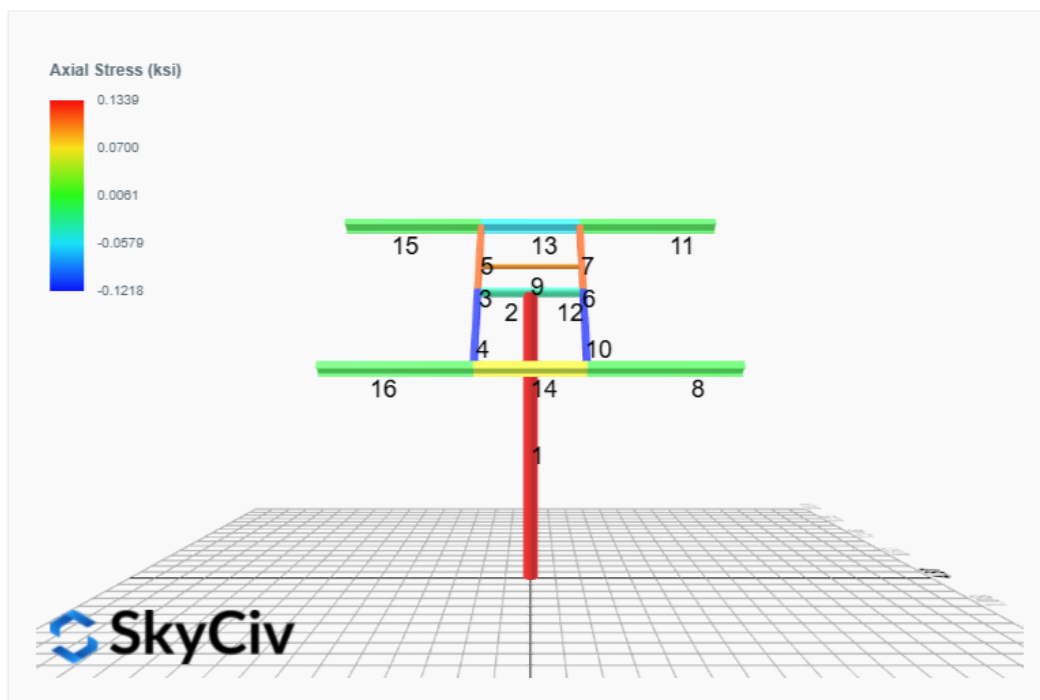




FEM Results (Envelope Worst Case for each member)







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	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 2. D + L	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 3. D + (S or Lr or R)	0.0000	2.8966	0.0000	0.0000	-0.0000	0.0178
ULS: 3. D + (S or Lr or R)	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.6152	0.0000	0.0000	-0.0000	0.0172
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 5b. D + 0.7E	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.6152	0.0000	0.0000	-0.0000	0.0172
ULS: 8. 0.6D + 0.7E	0.0000	1.0626	0.0000	0.0000	-0.0000	0.0093
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5096	3.8768	0.0000	0.0000	-0.0000	27.6066
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5096	-0.3348	0.0000	0.0000	-0.0000	-26.6657
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8822	4.1946	0.0000	0.0000	-0.0000	20.7105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.6152	0.0000	0.0000	-0.0000	0.0172
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8822	1.0359	0.0000	0.0000	-0.0000	-19.9937
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.6152	0.0000	0.0000	-0.0000	0.0172
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8822	3.3504	0.0000	0.0000	-0.0000	20.7088
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8822	0.1916	0.0000	0.0000	-0.0000	-19.9954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.7710	0.0000	0.0000	-0.0000	0.0155
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5096	3.1684	0.0000	0.0000	-0.0000	27.6004
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0626	0.0000	0.0000	-0.0000	0.0093
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5096	-1.0432	0.0000	0.0000	-0.0000	-26.6719
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0626	0.0000	0.0000	-0.0000	0.0093

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	6.1977
Shear X	-4.1827
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	46.6606

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	4.1946
Shear X	-2.5096
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	27.6066

Project Details

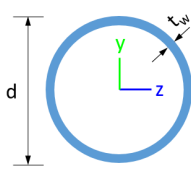
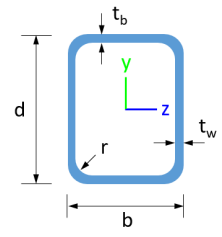
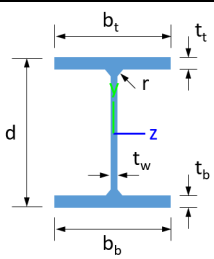
Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Youngla
 Unit System: imperial



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_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

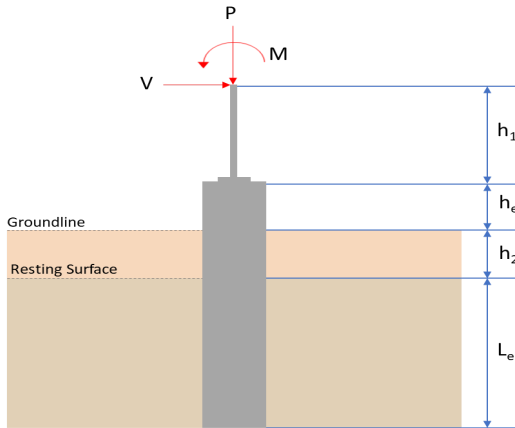
14	120.60	84.03	18.33	6.45	30.09	45.74
15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	23.36	6.45	30.09	45.74

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	0.050	0.750	0.000	0.037	0.000	0.775	#13	0.621	Not Required	Pass
2	0.002	0.303	0.257	0.068	0.049	0.560	#13	0.034	Not Required	Pass
3	0.009	0.592	0.066	0.059	0.010	0.639	#13	0.044	Not Required	Pass
4	0.008	0.590	0.143	0.059	0.025	0.657	#13	0.078	Not Required	Pass
5	0.009	0.368	0.148	0.059	0.029	0.391	#13	0.073	Not Required	Pass
6	0.009	0.592	0.066	0.059	0.010	0.639	#13	0.044	Not Required	Pass
7	0.009	0.368	0.148	0.059	0.029	0.391	#13	0.073	Not Required	Pass
8	0.000	0.069	0.094	0.028	0.007	0.143	#21	Not Required	Not Required	Pass
9	0.007	0.041	0.055	0.001	0.000	0.098	#13	0.198	Not Required	Pass
10	0.008	0.590	0.143	0.059	0.025	0.657	#13	0.078	Not Required	Pass
11	0.000	0.069	0.094	0.028	0.007	0.143	#21	Not Required	Not Required	Pass
12	0.002	0.303	0.257	0.068	0.049	0.560	#13	0.034	Not Required	Pass
13	0.004	0.197	0.202	0.042	0.010	0.337	#21	0.177	Not Required	Pass
14	0.005	0.201	0.202	0.042	0.010	0.337	#21	0.177	Not Required	Pass
15	0.000	0.069	0.094	0.028	0.007	0.143	#21	Not Required	Not Required	Pass
16	0.000	0.069	0.094	0.028	0.007	0.143	#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
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	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 = 9 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <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> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>4.195</td><td>6.198</td></tr><tr><td>Vx (kip)</td><td>-2.510</td><td>-4.183</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>27.607</td><td>46.661</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	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)	4.195	6.198	Vx (kip)	-2.510	-4.183	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	27.607	46.661	
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)	4.195	6.198																										
Vx (kip)	-2.510	-4.183																										
Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	0.000																										
Mz (kipft)	27.607	46.661																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load H = h1 + h2 + he H = (0 ft) + (0 ft) + (0 ft) H = 0 ft Considering x-direction: Ho - Lateral force per length of pile, Ho = Vz / D Ho = (-2.51 kip) / (36 in) Ho = -0.83667 kip/ft Mo - Moment per length of pile,																											

	$M_o = \frac{M_z + (V_x H)}{D}$ $M_o = \frac{(27.607 \text{ kipft}) + ((-2.51 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 9.2023 \text{ kipft/ft}$ 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: $L_{e,x} = 8.0495 \text{ ft}$ - 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, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(8.0495 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 8.049 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (9 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 9 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(8.049 \text{ ft})}{(9 \text{ ft})}$ $Ratio = 0.89433$	Status: PASS Ratio: 0.890
	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 $q = \frac{P_v}{A}$ $q = \frac{(4.195 \text{ kip})}{(7.0686 \text{ ft}^2)}$ $q = 0.59347 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.59347 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.29674$	Status: PASS Ratio: 0.300
Czeraniak	Lateral Soil Pressure (ASD):	

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(9 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 3$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.83667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.2023 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.2023 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (9.2023 \text{ kipft/ft})) + (4 \times (-0.83667 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2647 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (9.2023 \text{ kipft/ft})) + (3 \times (-0.83667 \text{ kip/ft}) \times (9 \text{ ft}))]^2}{(9 \text{ ft})^2 \times [(3 \times (9.2023 \text{ kipft/ft})) + (2 \times (-0.83667 \text{ kip/ft}) \times (9 \text{ ft}))]}$$

$$p = 0.23436 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (9.2023 \text{ kipft/ft})) + ((-0.83667 \text{ kip/ft}) \times (9 \text{ ft}))]}{(9 \text{ ft})^2}$$

$$s = 1.2654 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2647 \text{ ft})}{2}$$

$$p_a = 0.46985 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23436 \text{ kip/ft}^2)}{(0.46985 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.49879$$

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

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

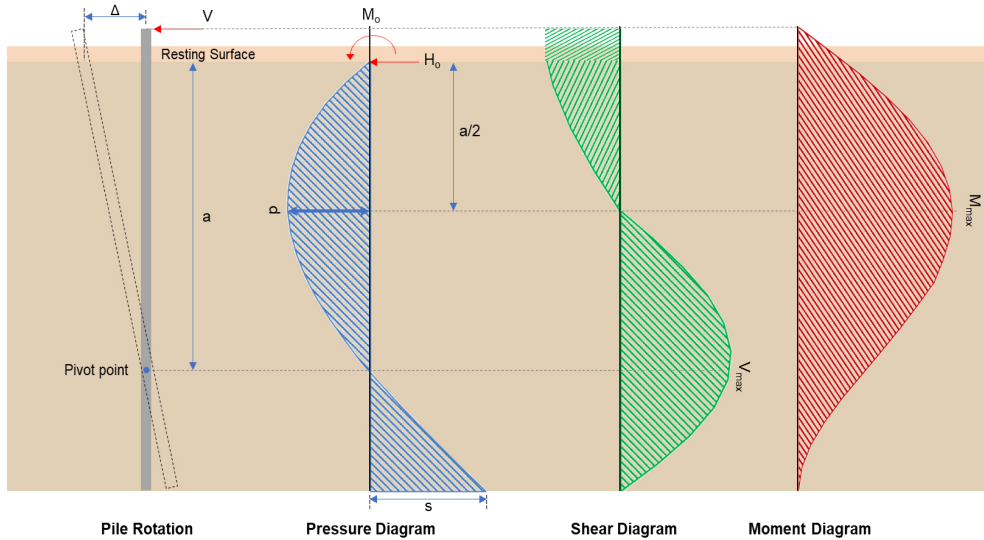
$$\text{Ratio} = \frac{(1.2654 \text{ kip/ft}^2)}{(1.35 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.500**

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

$$\text{Ratio} = 0.9373$$

Status: **PASS**
Ratio: **0.940**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.183 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.3943 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.661 \text{ kipft}) + ((-4.183 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 15.554 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.554 \text{ kipft/ft})}{(-1.3943 \text{ kip/ft})}$$

$$E = 11.155 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.554 \text{ kipft/ft}) \times (9 \text{ ft})) + (3 \times (-1.3943 \text{ kip/ft}) \times (9 \text{ ft})^2)}{(6 \times (15.554 \text{ kipft/ft})) + (4 \times (-1.3943 \text{ kip/ft}) \times (9 \text{ ft}))}$$

$$a = 6.2623 \text{ ft}$$

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

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

$$V_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.155 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2623 \text{ ft})}{(9 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.155 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2623 \text{ ft})}{(9 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 11.933 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-1.3943 \text{ kip/ft}) \times (36 \text{ in}) \times (9 \text{ ft})) \times \left[\left(\frac{(11.155 \text{ ft})}{(9 \text{ ft})} + \frac{(6.2623 \text{ ft})}{2 \times (9 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.155 \text{ ft})}{(9 \text{ ft})} + 3 \right) \times \left(\frac{(6.2623 \text{ ft})}{2 \times (9 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (11.155 \text{ ft})}{(9 \text{ ft})} + 2 \right) \times \left(\frac{(6.2623 \text{ ft})}{2 \times (9 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 50.297 \text{ kipft}$	
Table 22.4.2.1 22.4.2.2, 10.6.1.1	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, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(6.198 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -37.18 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-37.18 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ 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 $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $\text{Ratio} = 0.99533$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement,	Status: PASS Ratio: 1.000

	$s_{rebar} = Max [1.5, (1.5 d_{bar})]$ $s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10e: 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	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$ $\phi P_N = 1253.9 \text{ kip}$ <i>Ratio - Capacity</i> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(6.198 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0049429$	Status: PASS Ratio: 0.000
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 36 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$ λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$ 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 $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.198 \text{ kip} \rightarrow 6198 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(6198 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.49 \text{ kip}$$

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

$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.49 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.49 \text{ kip}$$

22.5.1.2 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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((75.49 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.879 \text{ kip}$$

Considering x-direction:

$V_{max} = 11.933 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(11.933 \text{ kip})}{(73.879 \text{ kip})}$$

$$Ratio = 0.16152$$

Status: **PASS**
 Ratio: **0.162**

		Ratio: 0.100
	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 $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 527.23 \text{ kipft}$ 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: $M_{max} = 50.297 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(50.297 \text{ kipft})}{(62.027 \text{ kipft})}$ $\text{Ratio} = 0.81089$	Status: PASS Ratio: 0.810