

Project Details



Project Name: john rutledge

Date: Mon Mar 31 2025

Location: 31944 CA-88, Pioneer, CA 95666, USA

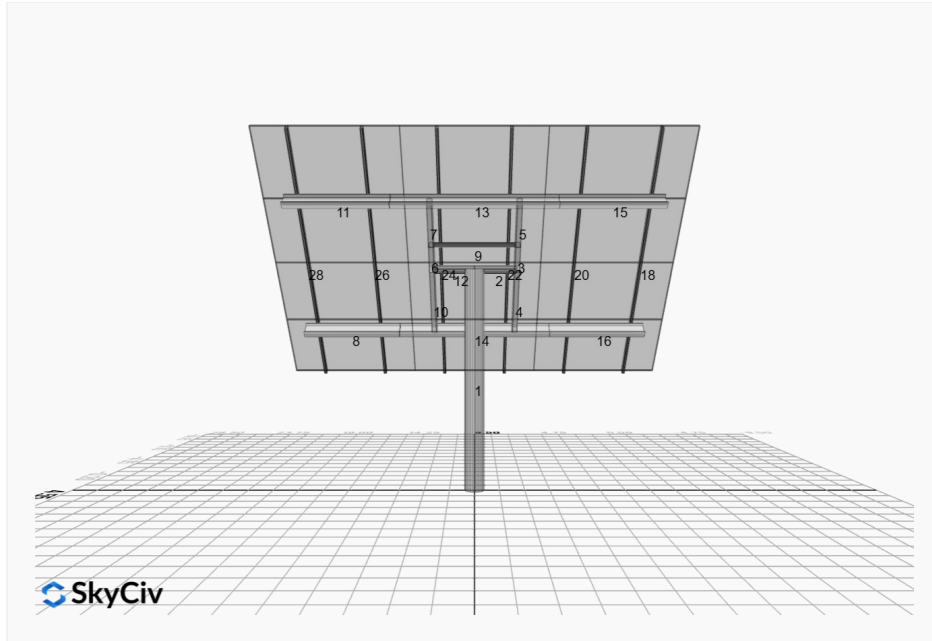
Number of Modules: 12

Unique ID: 1P-0-10TOP-HD-57-L-4Hx3W-D6I2

Number of Poles: 1

Dealer: _____

Date Sold: _____



Array Dimensions N/S	14.17 ft
Array Dimensions E/W	19.00 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

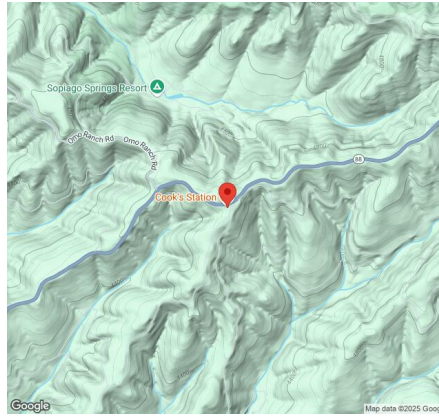
MT Solar Bill of Materials (1P-0-10TOP-HD-57-L-4Hx3W-D6I2)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-57	57IN HD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (170in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12
Ground Lug	3

Site Details:



Site Address: 31944 CA-88, Pioneer, CA 95666, USA

Array Specification

Duty Classification:	HD
Module Width:	42.00 in
Module Length:	75.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	15.85 ft
Total Frame Length:	17.00 ft
Module Info/Notes:	q cell
Array Dimensions N/S:	14.17 ft
Array Dimensions E/W:	19.00 ft
Rail Length:	170.00 in
Rail Spacing:	3.17 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	10.43 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: 7.00 ft
Foundation Volume:	4.148 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	31944 CA-88, Pioneer, CA 95666, USA
Wind Speed:	95 mph
Snow Load:	100 psf

Design Disclaimer

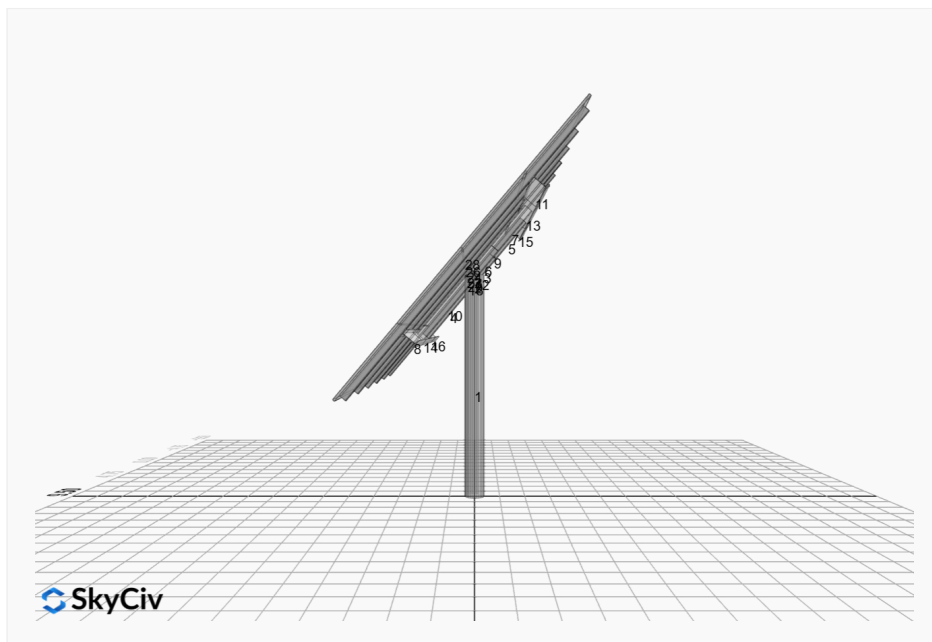
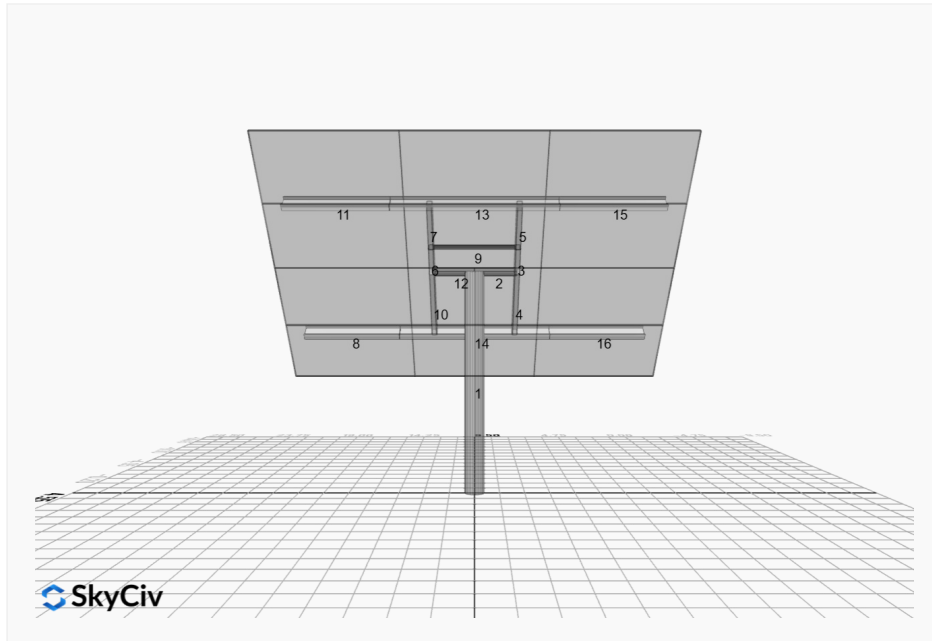
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

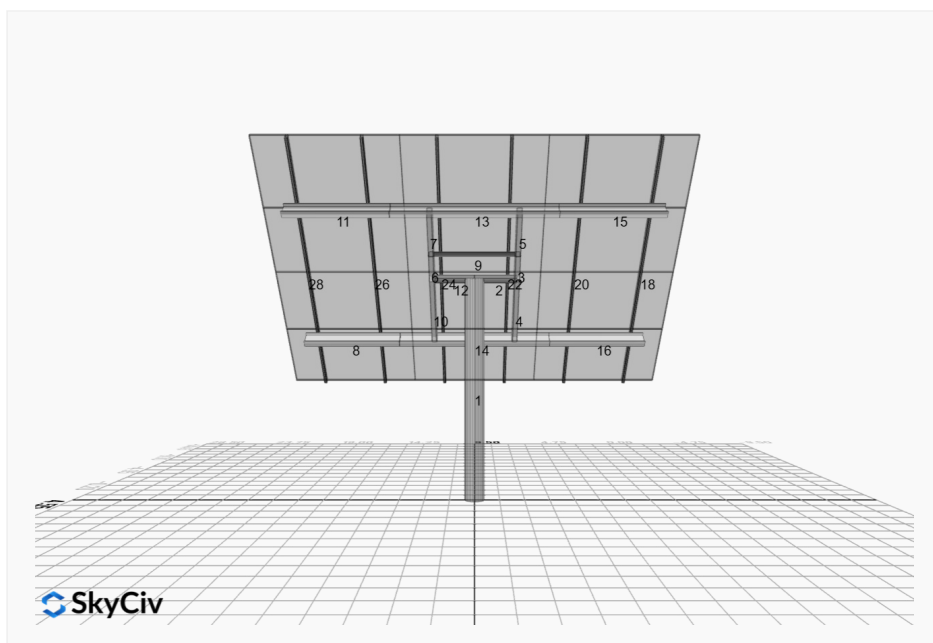
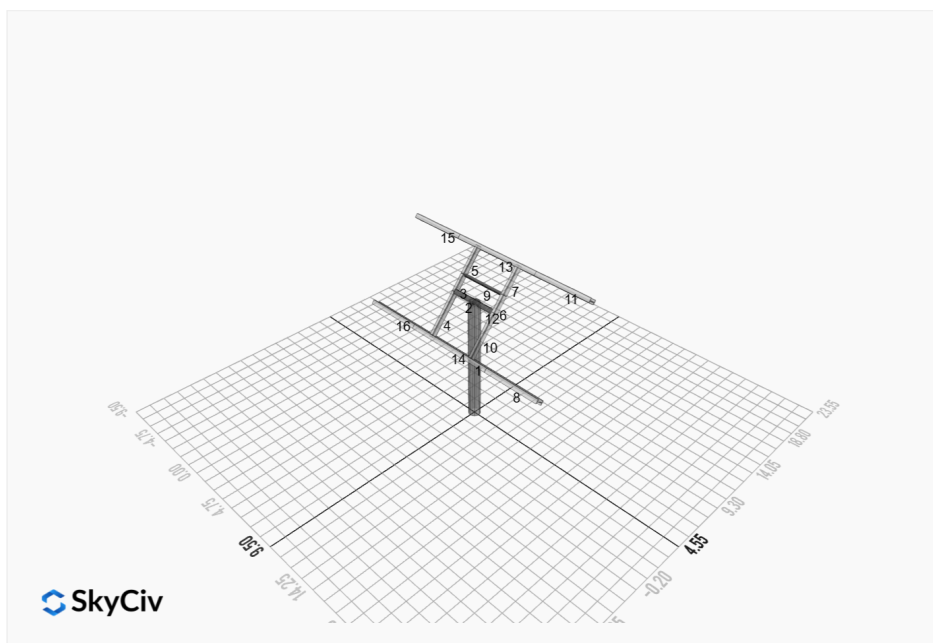
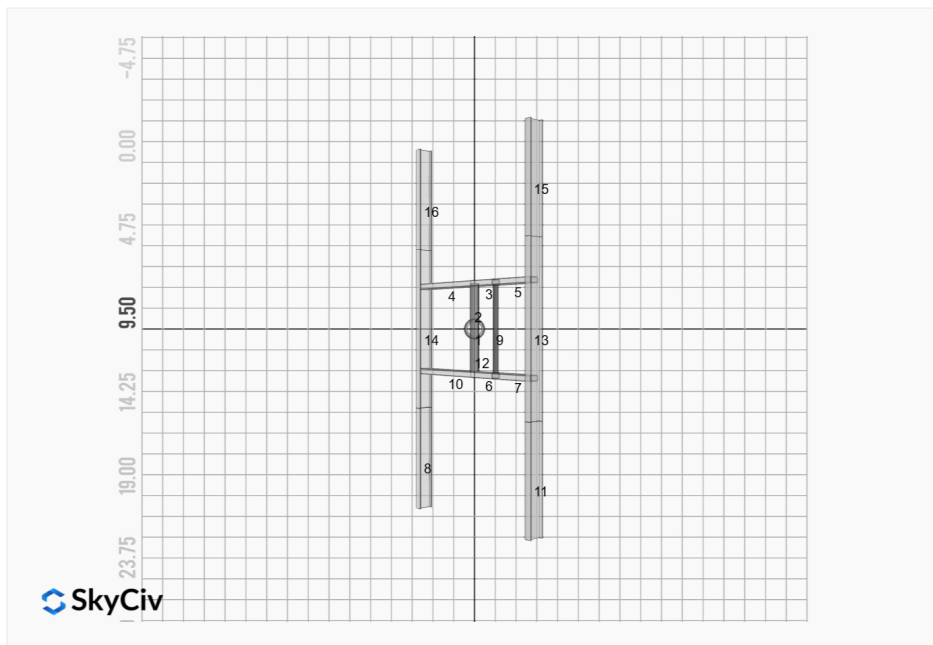
AutoDesigner Input

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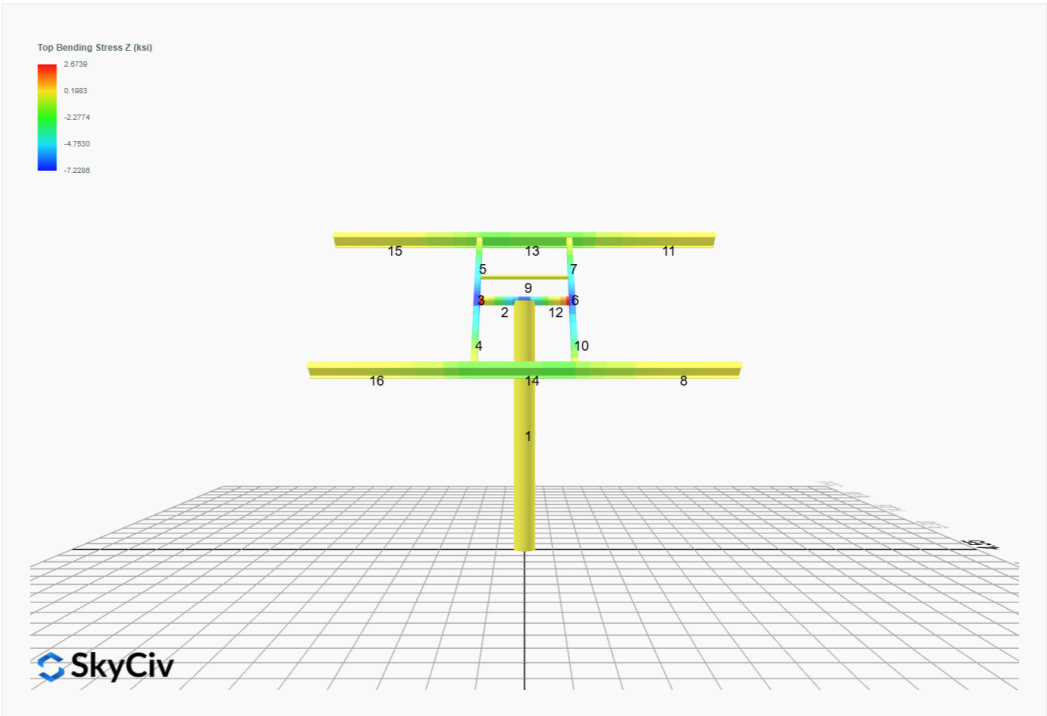
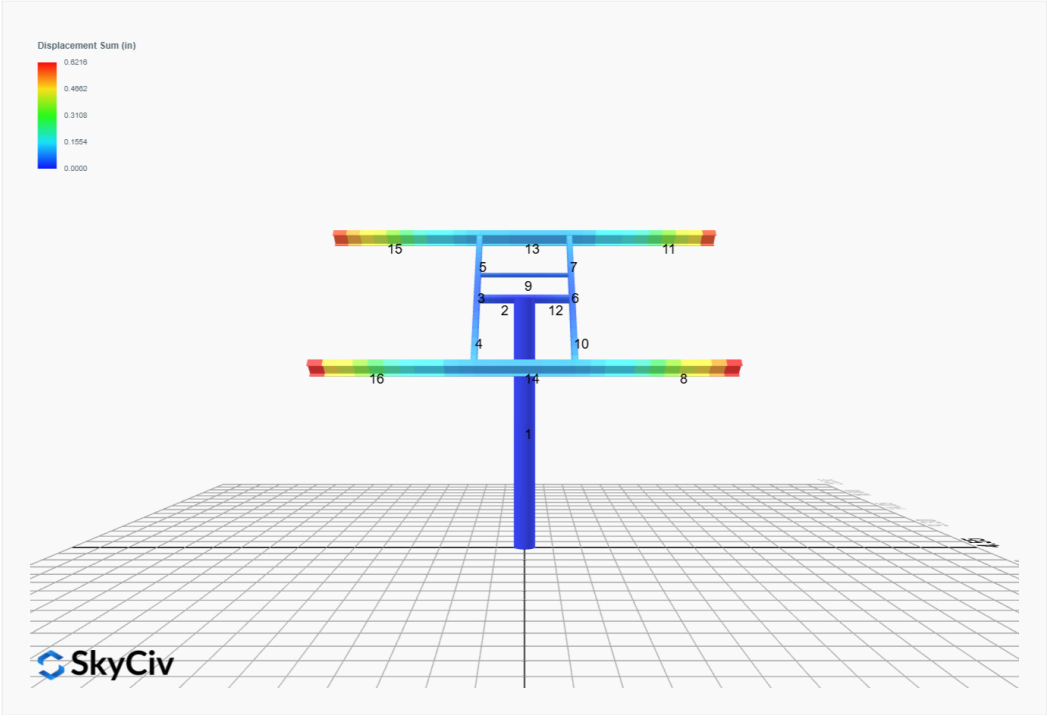
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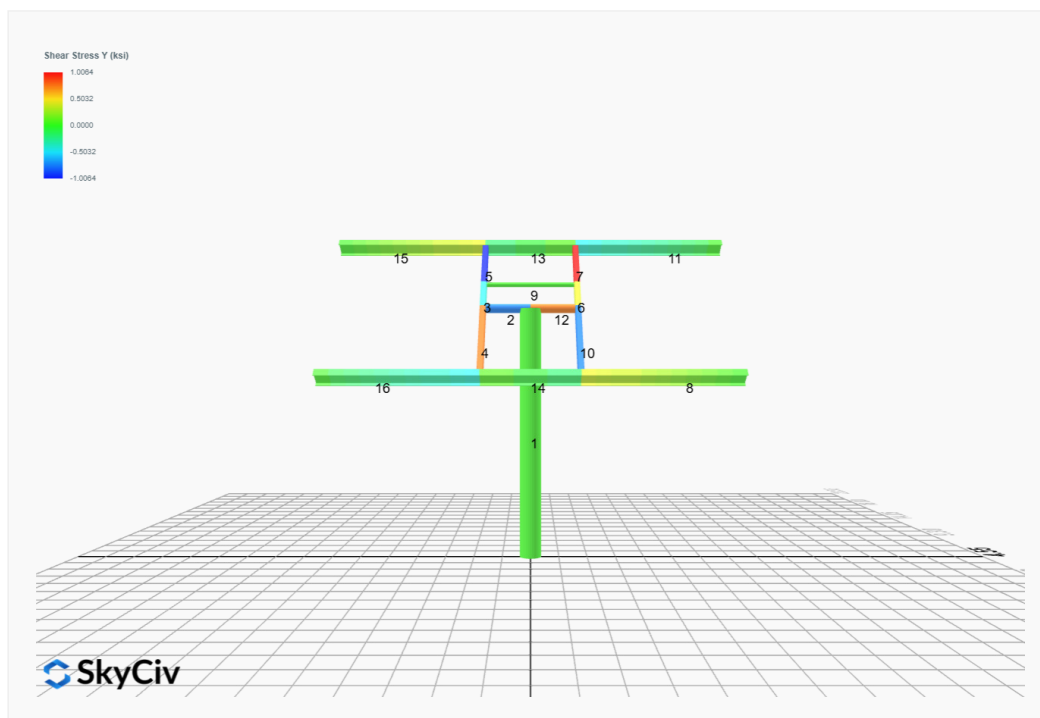
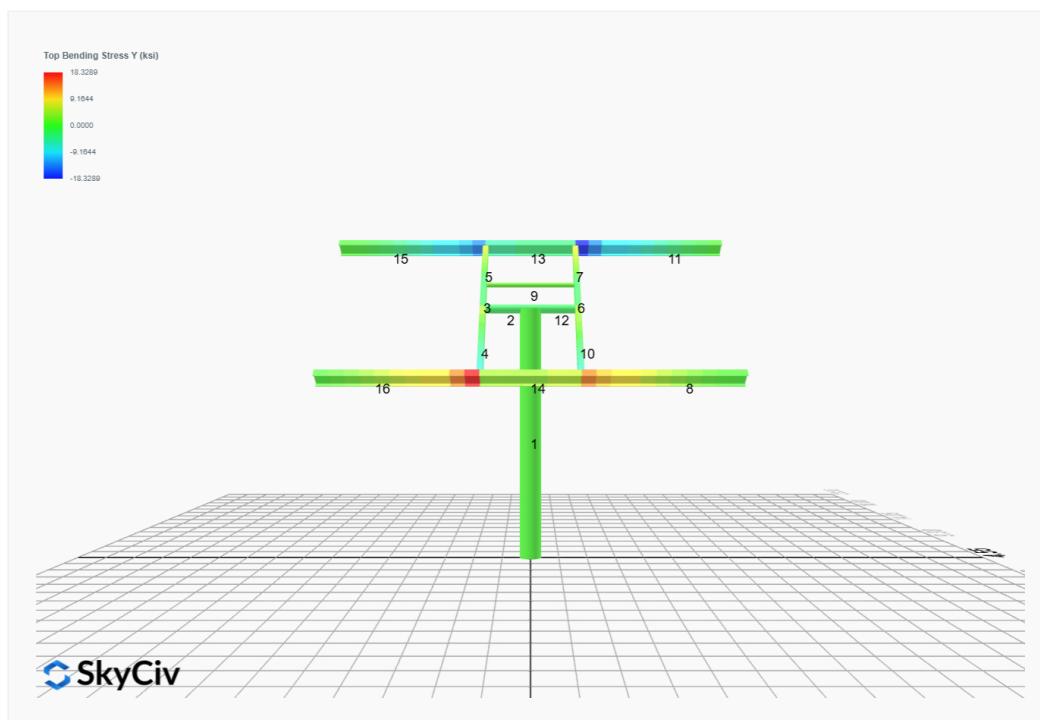
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

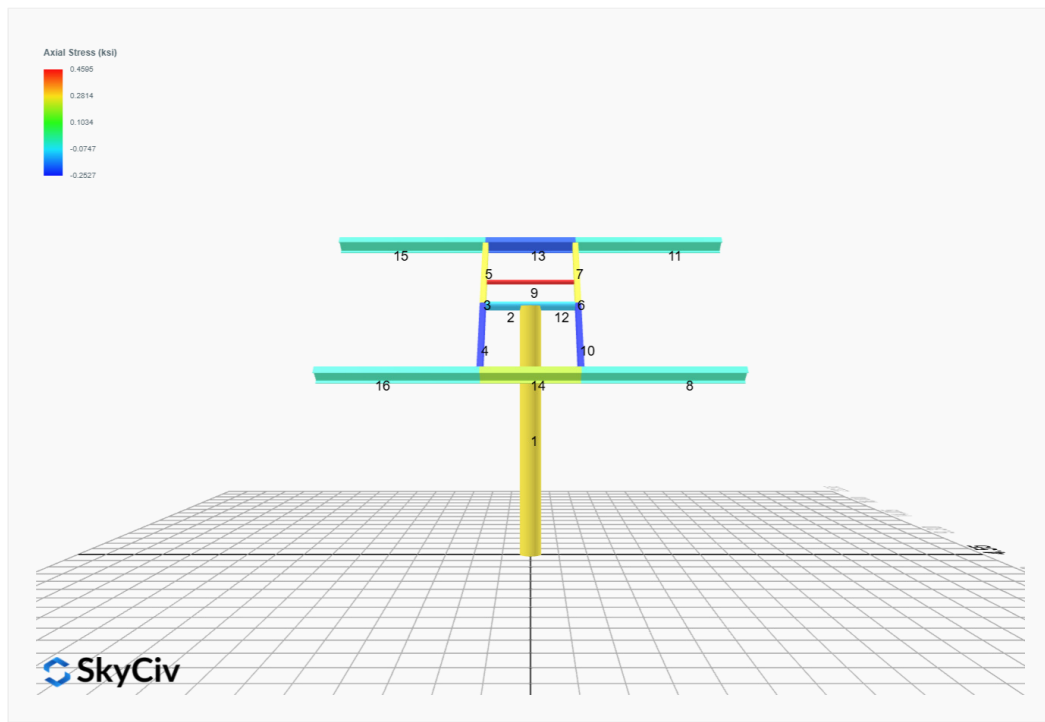




FEM Results (Envelope Worst Case for each member)







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	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 2. D + L	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 3. D + (S or Lr or R)	0.0000	5.6380	0.0000	0.0000	-0.0000	0.0352
ULS: 3. D + (S or Lr or R)	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.7868	0.0000	0.0000	-0.0000	0.0316
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 5b. D + 0.7E	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.7868	0.0000	0.0000	-0.0000	0.0316
ULS: 8. 0.6D + 0.7E	0.0000	1.3400	0.0000	0.0000	-0.0000	0.0123
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.4846	5.1573	0.0000	0.0000	-0.0000	36.5512
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.4846	-0.6906	0.0000	0.0000	-0.0000	-36.1137
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6135	6.9798	0.0000	0.0000	-0.0000	27.4296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.7868	0.0000	0.0000	-0.0000	0.0316
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6135	2.5939	0.0000	0.0000	-0.0000	-27.0692
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.7868	0.0000	0.0000	-0.0000	0.0316
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6135	4.4263	0.0000	0.0000	-0.0000	27.4185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6135	0.0404	0.0000	0.0000	-0.0000	-27.0802
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.2334	0.0000	0.0000	-0.0000	0.0205
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.4846	4.2640	0.0000	0.0000	-0.0000	36.5430
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.3400	0.0000	0.0000	-0.0000	0.0123
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.4846	-1.5839	0.0000	0.0000	-0.0000	-36.1220
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.3400	0.0000	0.0000	-0.0000	0.0123

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	10.5640
Shear X	-5.8077
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	61.2832

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	6.9798
Shear X	-3.4846
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	36.5512

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: john rutledge
 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					
11	10in Pipe Sch 40	10.75	0.36					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x16	5.00	3.00	0.17	0.17	0.17		
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_{yD} (in ⁴)	I_{zD} (in ⁴)	I_w (in ⁶)	S_{yD} (in ³)	S_{zD} (in ³)

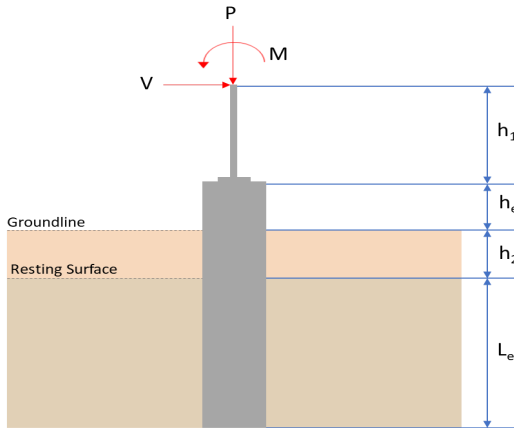
14	133.20	85.85	23.59	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62

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	0.029	0.415	0.000	0.036	0.000	0.428	#13	0.358	Not Required	Pass
2	0.006	0.349	0.265	0.085	0.049	0.594	#13	0.035	Not Required	Pass
3	0.013	0.595	0.062	0.060	0.002	0.631	#13	0.045	Not Required	Pass
4	0.012	0.592	0.240	0.059	0.050	0.670	#13	0.080	Not Required	Pass
5	0.013	0.369	0.249	0.059	0.063	0.402	#13	0.074	Not Required	Pass
6	0.013	0.595	0.062	0.060	0.002	0.631	#13	0.045	Not Required	Pass
7	0.013	0.369	0.249	0.059	0.063	0.402	#13	0.074	Not Required	Pass
8	0.000	0.100	0.306	0.034	0.018	0.392	#21	Not Required	Not Required	Pass
9	0.024	0.040	0.058	0.001	0.000	0.103	#13	0.204	Not Required	Pass
10	0.012	0.592	0.240	0.059	0.050	0.670	#13	0.080	Not Required	Pass
11	0.000	0.100	0.306	0.034	0.018	0.392	#21	Not Required	Not Required	Pass
12	0.006	0.349	0.265	0.085	0.049	0.594	#13	0.035	Not Required	Pass
13	0.011	0.274	0.573	0.047	0.025	0.797	#21	0.190	Not Required	Pass
14	0.013	0.278	0.573	0.047	0.025	0.797	#21	0.190	Not Required	Pass
15	0.000	0.100	0.306	0.034	0.018	0.392	#21	Not Required	Not Required	Pass
16	0.000	0.100	0.306	0.034	0.018	0.392	#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: rectangular b = 48 in - Pile width D = 48 in - Pile depth L = 7 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>6.980</td><td>10.564</td></tr><tr><td>Vx (kip)</td><td>-3.485</td><td>-5.808</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>36.551</td><td>61.283</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)	6.980	10.564	Vx (kip)	-3.485	-5.808	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	36.551	61.283	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 \, D}$ $H_o = \frac{(-3.485 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.55494 \text{ kip/ft}$	
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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Vx (kip)	-3.485	-5.808																											
Vz (kip)	0.000	0.000																											
Mx (kipft)	0.000	0.000																											
Mz (kipft)	36.551	61.283																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(36.551 \text{ kipft}) + ((-3.485 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 5.8202 \text{ kipft/ft}$ 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: $L_{e,x} = 6.3379 \text{ ft}$ - Required depth in x-direction, Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.3379 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.338 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.338 \text{ ft})}{(7 \text{ ft})}$ $\text{Ratio} = 0.90543$	Status: PASS Ratio: 0.910
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_v}{A}$ $q = \frac{(6.98 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.43625 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.43625 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.21813$	Status: PASS Ratio: 0.220
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(7 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.75$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.55494 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.8202 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8202 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.55494 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (5.8202 \text{ kipft/ft})) + (4 \times (-0.55494 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8463 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.8202 \text{ kipft/ft})) + (3 \times (-0.55494 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (5.8202 \text{ kipft/ft})) + (2 \times (-0.55494 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.21351 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.8202 \text{ kipft/ft})) + ((-0.55494 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.9497 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8463 \text{ ft})}{2}$$

$$p_a = 0.36347 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21351 \text{ kip/ft}^2)}{(0.36347 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.58743$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

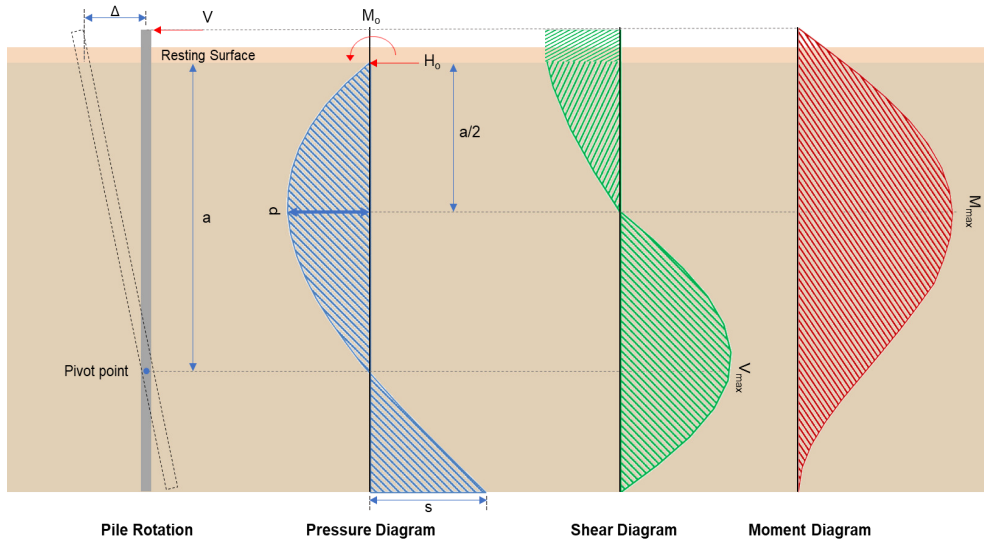
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.9497 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.590**

$$Ratio = 0.90448$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.808 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.92484 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.283 \text{ kipft}) + ((-5.808 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.7584 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.7584 \text{ kipft/ft})}{(-0.92484 \text{ kip/ft})}$$

$$E = 10.551 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.7584 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.92484 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (9.7584 \text{ kipft/ft})) + (4 \times (-0.92484 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8455 \text{ ft}$$

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

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

$$V_{max} = ((-0.92484 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.551 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8455 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.551 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8455 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = -19.908 \text{ kips}$$

	$V_{max} = 12.500 \text{ kip}$ M_{max} - Max bending moment located at depth a/2, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.92484 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(10.551 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8455 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.551 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8455 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.551 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8455 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 40.726 \text{ kipft}$	
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, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(10.564 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.245 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.245 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$	Status: PASS Ratio: 0.970

25.7.2.2 25.7.2.1	$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) s_{ties} - Maximum spacing of ties, $s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), Min (D, b)]$ $s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min ((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #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 $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(10.564 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0039489$	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 = 48 in - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ 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 $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.564 \text{ kip} \rightarrow 10564 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10564 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.89 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.89 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.89 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.89 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.01 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.306 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(12.306 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.11086$$

Status: **PASS**
Ratio: **0.110**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

$$\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 40.726 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(40.726 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.16316$$

Status: **PASS**
Ratio: **0.160**