

Project Details



Project Name: Hamed Adibnatanzi - 624 Sugarloaf

Court

Date: Wed Sep 24 2025

Location: 624 Sugarloaf Ct, Walnut Creek, CA 94596, USA

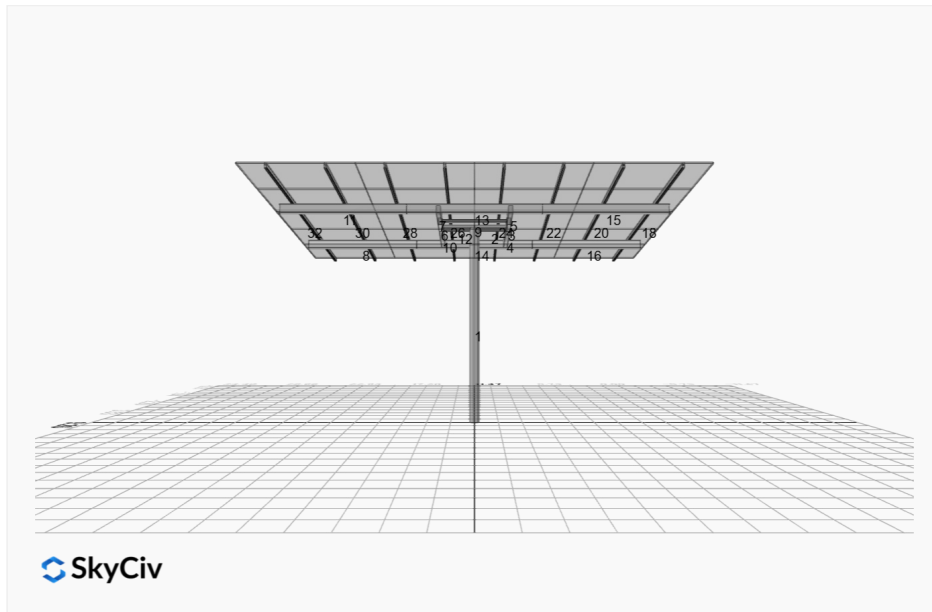
Number of Modules: 20

Number of Poles: 1

Unique ID: 1P-0-6TOP-SD-84-L-5Hx4W-CB29

Date Sold:

Dealer: _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	10
Front Edge Clearance	10 ft

MT Solar Bill of Materials (1P-0-6TOP-SD-84-L-5Hx4W-CB29)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-84	84IN SD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	4

Rail Bill of Materials

Part	Qty
Rails (226in)	8
Rail Attachment	32
Module Mid Clamp	32
Module End Clamp	16
Ground Lug	4

Site Details:



Site Address: 624 Sugarloaf Ct, Walnut Creek, CA 94596, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	67.80in
Number of Rows:	5
Number of Columns:	4
Total Number of Modules:	20
Winter Tilt Angle:	10
Front Edge Clearance:	10
Total Array Height at Tilt:	13.27 ft
Total Frame Length:	21.50 ft
Module Info/Notes:	Longi: LR5-54HPB-405M
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	22.93 ft
Rail Length:	225.73 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	11.63 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: 4.75 ft
Foundation Volume:	2.815 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	624 Sugarloaf Ct, Walnut Creek, CA 94596, USA
Wind Speed:	87 mph
Snow Load:	0 psf

Design Disclaimer

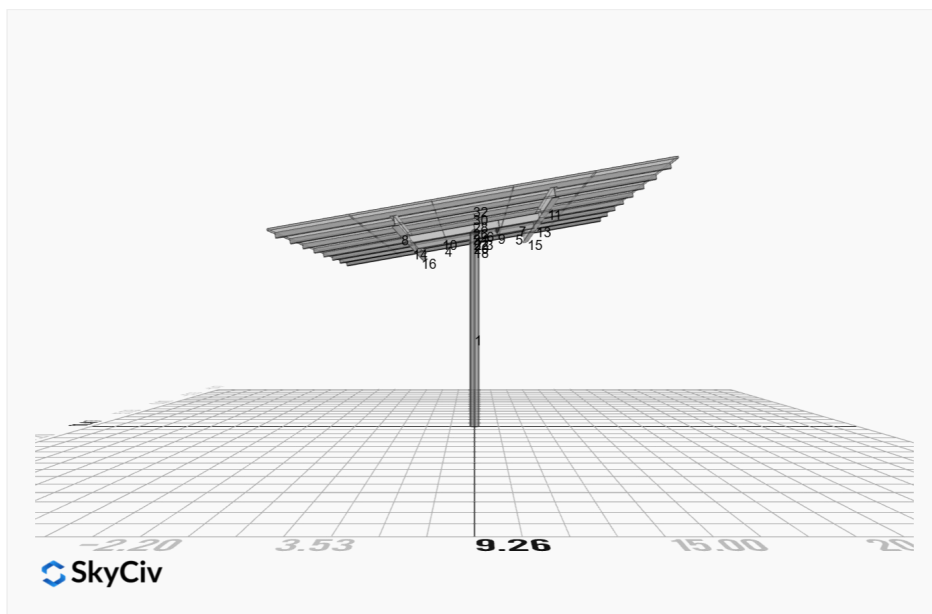
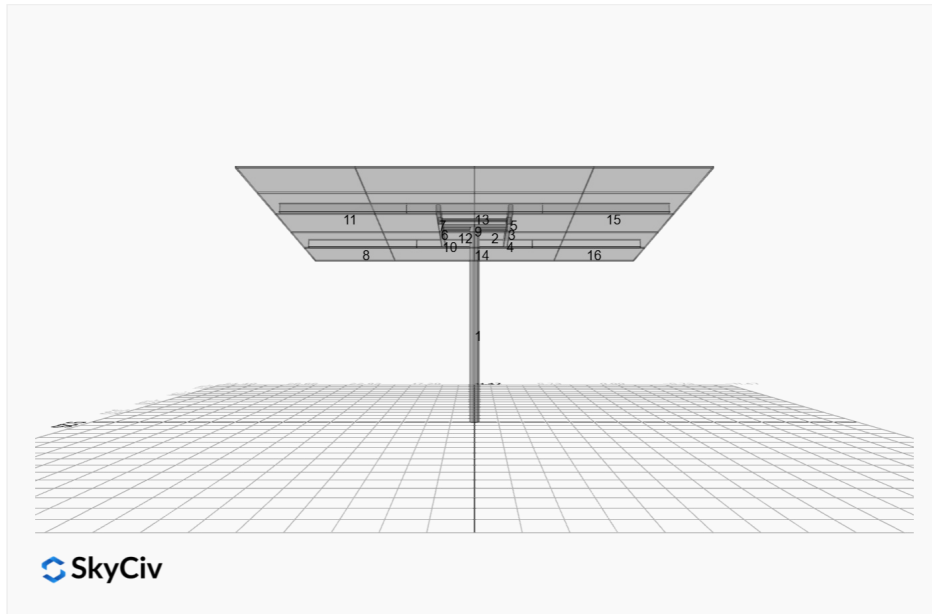
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

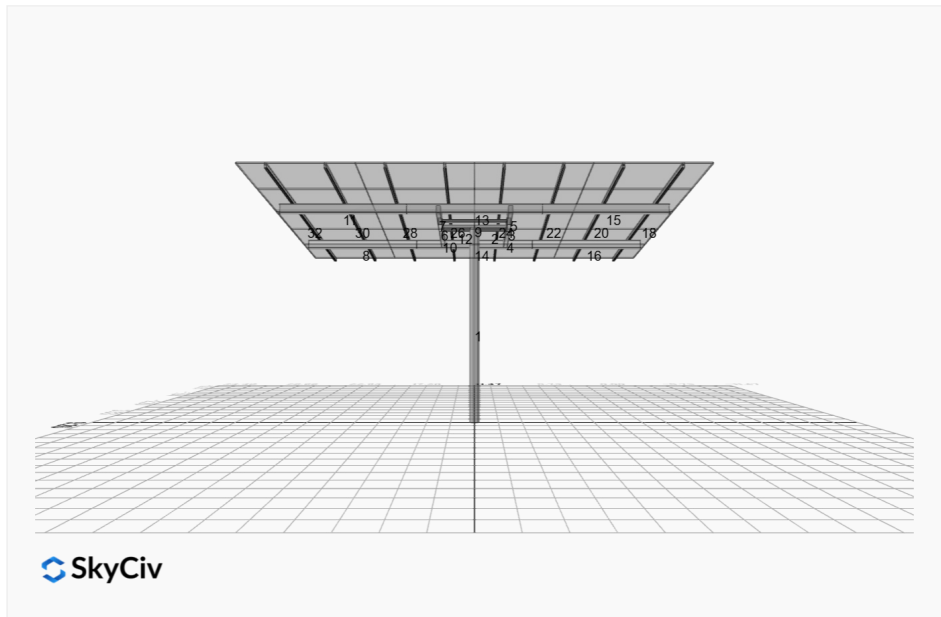
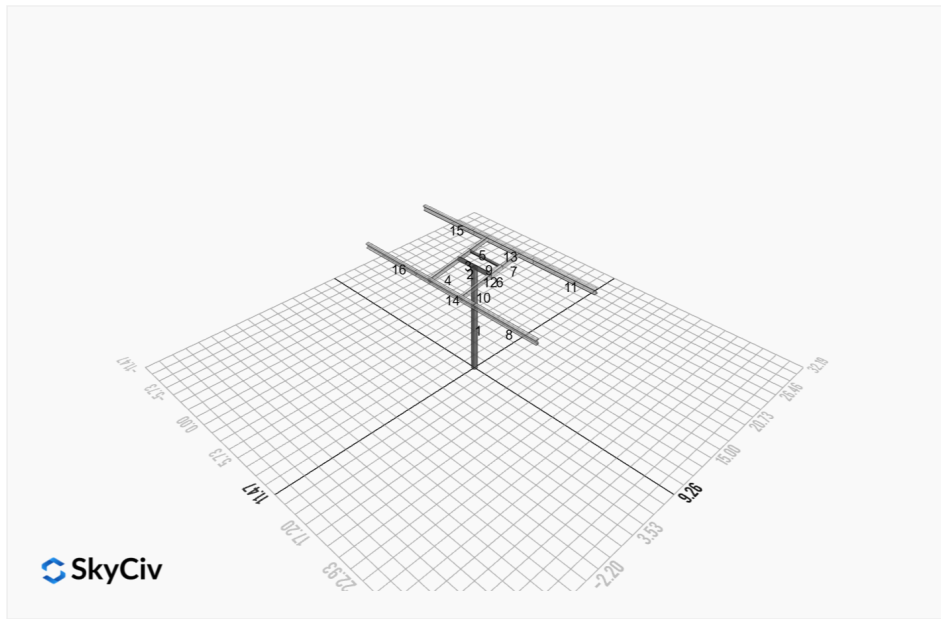
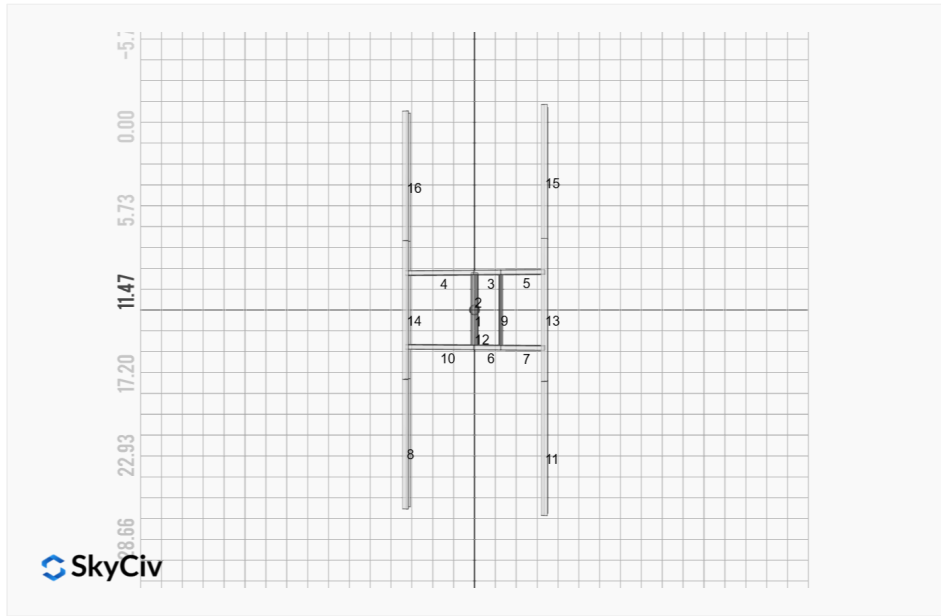
AutoDesigner Input

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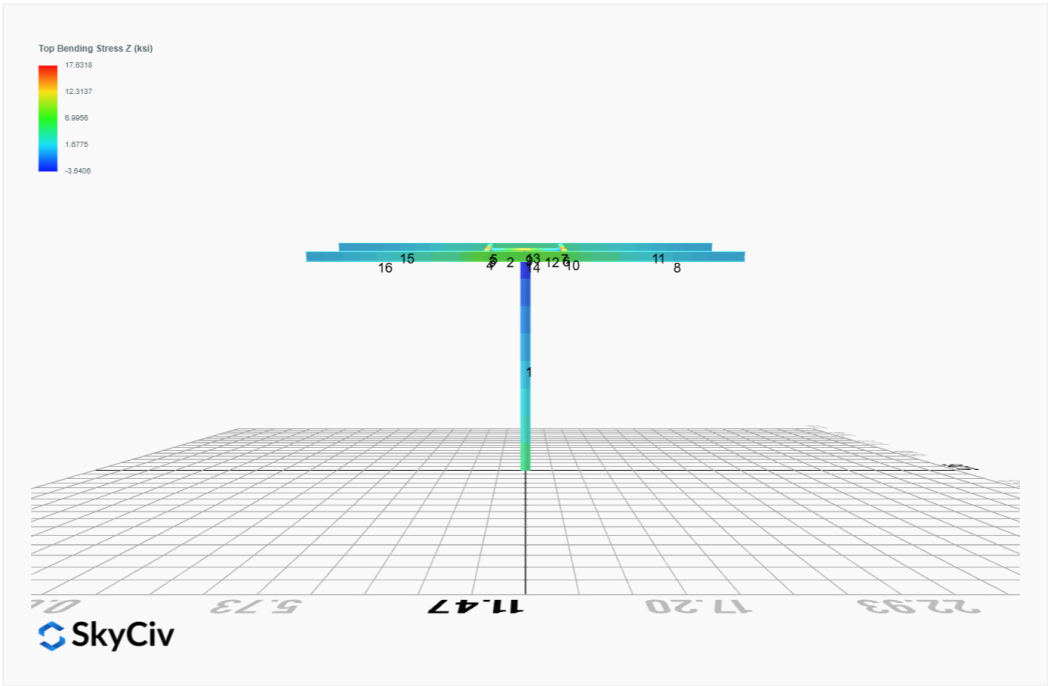
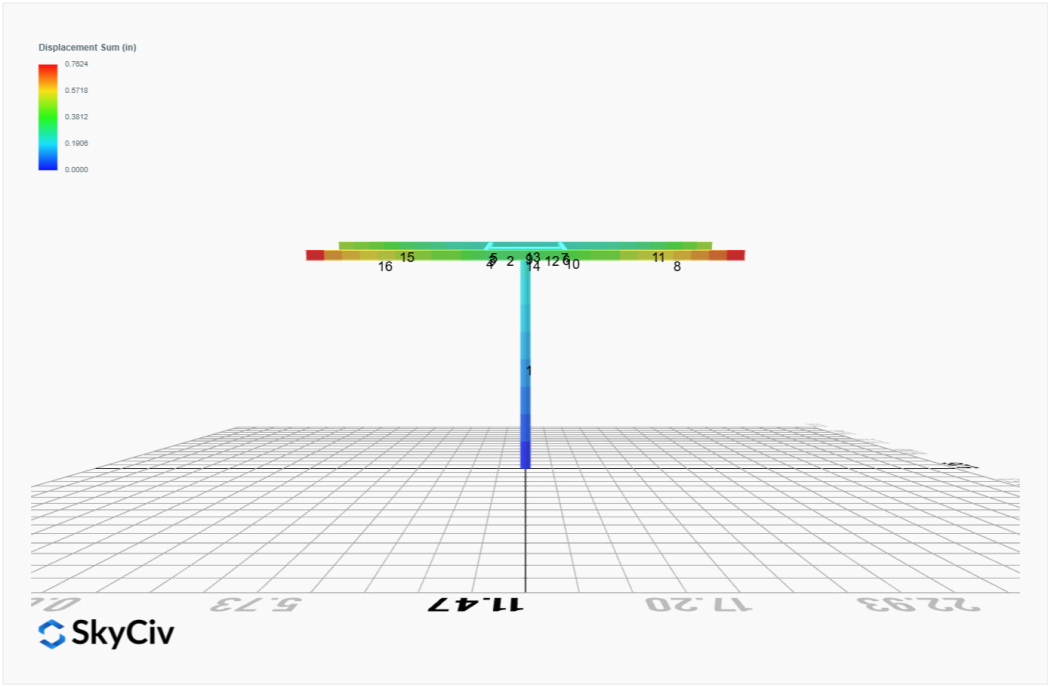
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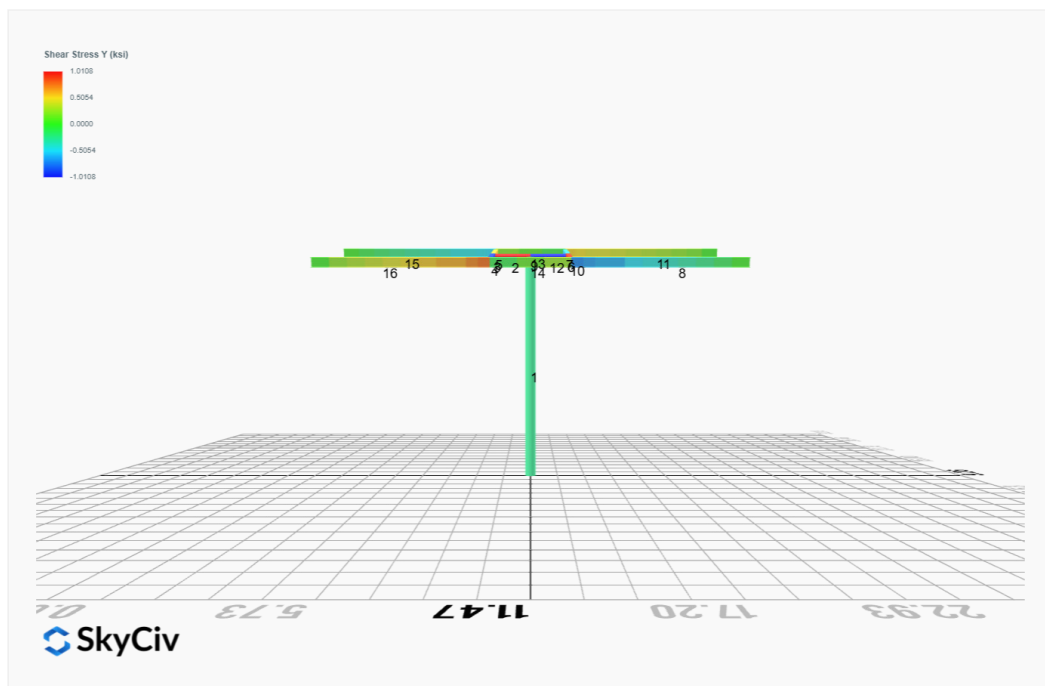
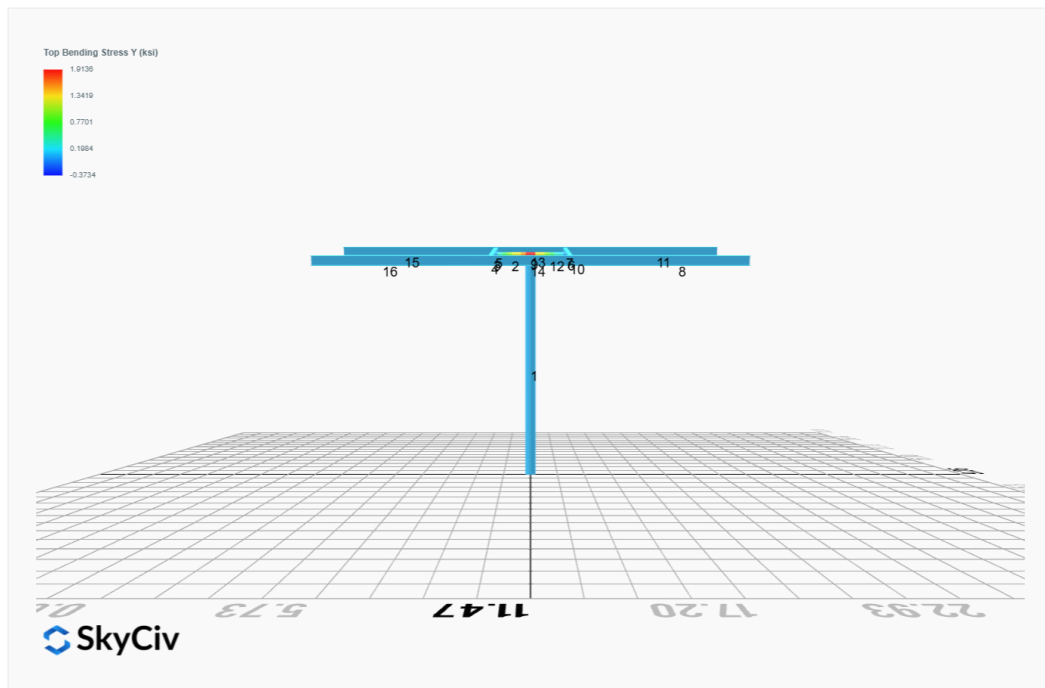
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

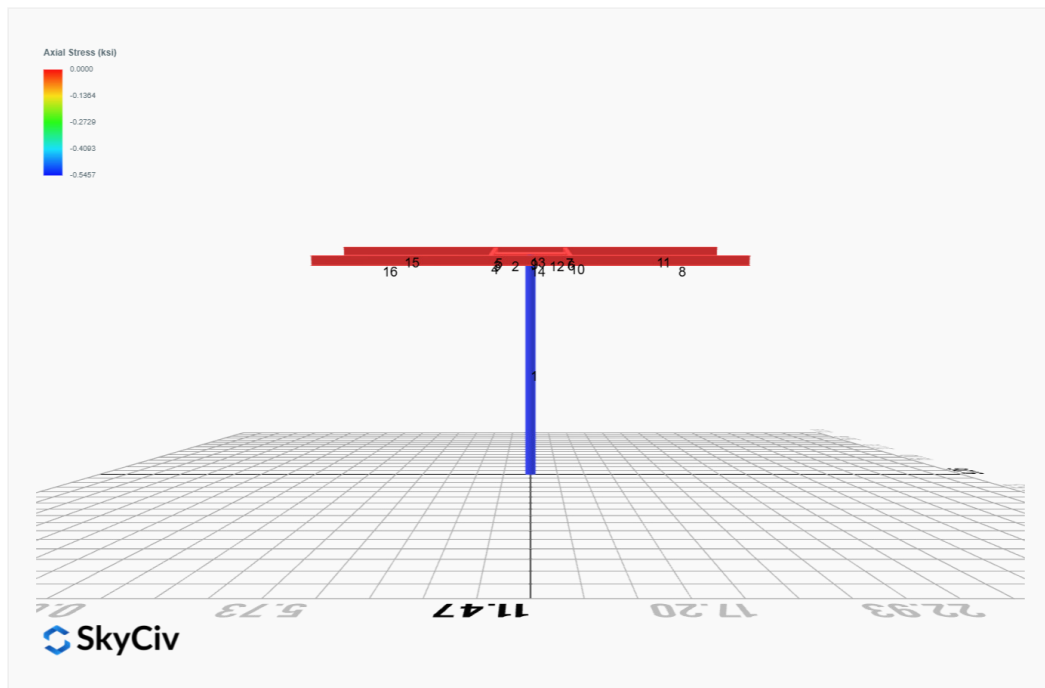




FEM Results (Envelope Worst Case for each member)







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	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 2. D + L	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 3. D + (S or Lr or R)	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 3. D + (S or Lr or R)	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 5b. D + 0.7E	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.7267	0.0000	0.0000	-0.0000	0.0232
ULS: 8. 0.6D + 0.7E	0.0000	1.6360	0.0000	0.0000	-0.0000	0.0139
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.4595	5.3327	0.0000	0.0000	-0.0000	7.5993
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.4595	5.3327	0.0000	0.0000	-0.0000	7.5993
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3222	0.8991	0.0000	0.0000	-0.0000	-2.1454
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2805	1.1360	0.0000	0.0000	-0.0000	-9.0427
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3446	4.6812	0.0000	0.0000	-0.0000	5.7053
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3446	4.6812	0.0000	0.0000	-0.0000	5.7053
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2417	1.3560	0.0000	0.0000	-0.0000	-1.6033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2104	1.5337	0.0000	0.0000	-0.0000	-6.7763
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3446	4.6812	0.0000	0.0000	-0.0000	5.7053
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3446	4.6812	0.0000	0.0000	-0.0000	5.7053
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2417	1.3560	0.0000	0.0000	-0.0000	-1.6033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2104	1.5337	0.0000	0.0000	-0.0000	-6.7763
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.4595	4.2420	0.0000	0.0000	-0.0000	7.5900
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.4595	4.2420	0.0000	0.0000	-0.0000	7.5900
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3222	-0.1915	0.0000	0.0000	-0.0000	-2.1547
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2805	0.0454	0.0000	0.0000	-0.0000	-9.0520

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	7.6153
Shear X	-0.7658
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	15.6107

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	5.3327
Shear X	-0.4595
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	9.0520

Project Details

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



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions								
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
7	6in Pipe Sch 40	6.63	0.28					
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)

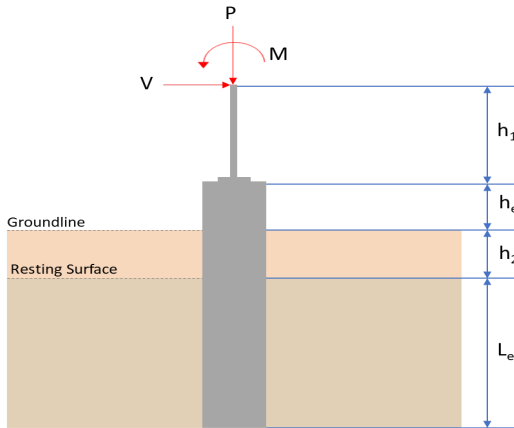
15	120.60	15.97	23.36	6.45	30.09	45.74
16	120.60	15.97	23.36	6.45	30.09	45.74

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	0.103	0.369	0.000	0.010	0.000	0.373	#16	0.653	Not Required	Pass
2	0.001	0.429	0.057	0.086	0.009	0.486	#13	0.034	Not Required	Pass
3	0.002	0.694	0.016	0.070	0.002	0.701	#13	0.044	Not Required	Pass
4	0.002	0.546	0.058	0.055	0.009	0.575	#13	0.078	Not Required	Pass
5	0.002	0.430	0.062	0.070	0.013	0.445	#13	0.073	Not Required	Pass
6	0.002	0.694	0.016	0.070	0.002	0.701	#13	0.044	Not Required	Pass
7	0.002	0.430	0.062	0.070	0.013	0.445	#13	0.073	Not Required	Pass
8	0.000	0.155	0.051	0.034	0.002	0.198	#13	Not Required	Not Required	Pass
9	0.006	0.092	0.018	0.001	0.000	0.112	#13	0.198	Not Required	Pass
10	0.002	0.546	0.058	0.055	0.009	0.575	#13	0.078	Not Required	Pass
11	0.000	0.197	0.051	0.044	0.002	0.240	#13	Not Required	Not Required	Pass
12	0.001	0.429	0.057	0.086	0.009	0.486	#13	0.034	Not Required	Pass
13	0.002	0.398	0.079	0.054	0.003	0.466	#13	0.177	Not Required	Pass
14	0.002	0.323	0.079	0.043	0.003	0.383	#13	0.177	Not Required	Pass
15	0.000	0.197	0.051	0.044	0.002	0.240	#13	Not Required	Not Required	Pass
16	0.000	0.155	0.051	0.034	0.002	0.198	#13	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 = 4.75 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>5.333</td><td>7.615</td></tr><tr><td>Vx (kip)</td><td>-0.460</td><td>-0.766</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>9.052</td><td>15.611</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)	5.333	7.615	Vx (kip)	-0.460	-0.766	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	9.052	15.611	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, $H_o = \frac{V_x}{1.57 \text{ } D}$ $H_o = \frac{(-0.46 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.073248 \text{ kip/ft}$	
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																										
Load Component	ASD	LRFD																											
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Vx (kip)	-0.460	-0.766																											
Vz (kip)	0.000	0.000																											
Mx (kipft)	0.000	0.000																											
Mz (kipft)	9.052	15.611																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(9.052 \text{ kipft}) + ((-0.46 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.4414 \text{ kipft/ft}$ 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: $L_{e,x} = 4.5668 \text{ ft}$ - 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, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(4.5668 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 4.567 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (4.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 4.75 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(4.567 \text{ ft})}{(4.75 \text{ ft})}$ $Ratio = 0.96147$	Status: PASS Ratio: 0.960
	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 $q = \frac{P_v}{A}$ $q = \frac{(5.333 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.33331 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.33331 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.16666$	Status: PASS Ratio: 0.170
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(4.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.073248 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.4414 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4414 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.073248 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (1.4414 \text{ kipft/ft})) + (4 \times (-0.073248 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2215 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.4414 \text{ kipft/ft})) + (3 \times (-0.073248 \text{ kip/ft}) \times (4.75 \text{ ft}))]^2}{(4.75 \text{ ft})^2 \times [(3 \times (1.4414 \text{ kipft/ft})) + (2 \times (-0.073248 \text{ kip/ft}) \times (4.75 \text{ ft}))]}$$

$$p = 0.20426 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.4414 \text{ kipft/ft})) + ((-0.073248 \text{ kip/ft}) \times (4.75 \text{ ft}))]}{(4.75 \text{ ft})^2}$$

$$s = 0.67409 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.2215 \text{ ft})}{2}$$

$$p_a = 0.24162 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20426 \text{ kip/ft}^2)}{(0.24162 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84539$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.75 \text{ ft})$$

$$p_s = 0.7125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

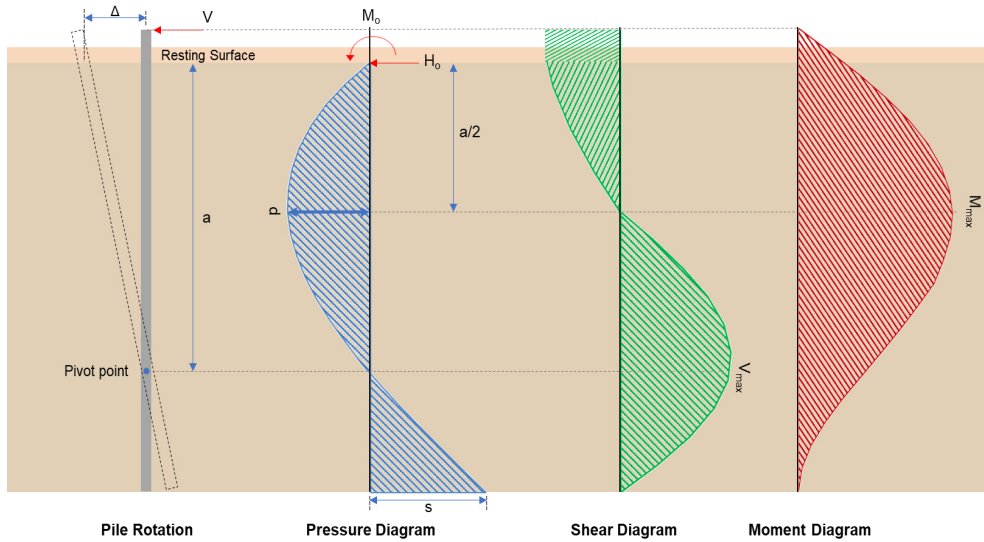
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.67409 \text{ kip/ft}^2)}{(0.7125 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.850**

$$Ratio = 0.9461$$

Status: **PASS**
Ratio: **0.950**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.766 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.12197 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.611 \text{ kipft}) + ((-0.766 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.4858 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.4858 \text{ kipft/ft})}{(-0.12197 \text{ kip/ft})}$$

$$E = 20.38 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4858 \text{ kipft/ft}) \times (4.75 \text{ ft})) + (3 \times (-0.12197 \text{ kip/ft}) \times (4.75 \text{ ft})^2)}{(6 \times (2.4858 \text{ kipft/ft})) + (4 \times (-0.12197 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2199 \text{ ft}$$

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

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

$$V_{max} = ((-0.12197 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (20.38 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2199 \text{ ft})}{(4.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (20.38 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2199 \text{ ft})}{(4.75 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 4.0323 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.12197 \text{ kip/ft}) \times (48 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(20.38 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2199 \text{ ft})}{2 \times (4.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (20.38 \text{ ft})}{(4.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.2199 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (20.38 \text{ ft})}{(4.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.2199 \text{ ft})}{2 \times (4.75 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 9.3643 \text{ kipft}$	
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, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(7.615 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.343 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.343 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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 $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.615 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0028465$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.615 \text{ kip} \rightarrow 7615 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7615 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.5 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.5 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.5 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.5 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.76 \text{ kip}$$

Considering x-direction:

$V_{max} = 4.0323 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(4.0323 \text{ kip})}{(110.76 \text{ kip})}$$

$$Ratio = 0.036407$$

Status: **PASS**
Ratio: **0.040**

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:

$M_{max} = 9.3643 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.3643 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.037517$$

Status: **PASS**
Ratio: **0.040**