

Project Details



Project Name: Cabven - 5x2 - 48round - V1Jb

Date: Mon Aug 05 2024

Location: Crested Butte, CO 81224, USA

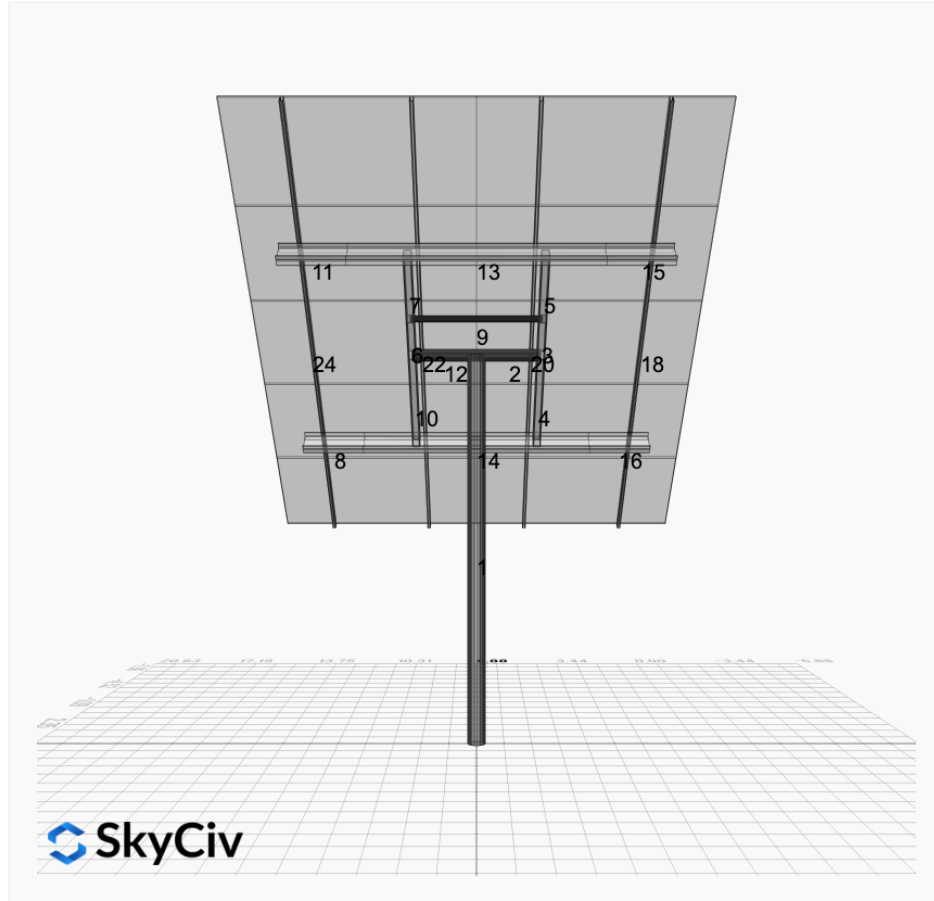
Number of Modules: 10

Unique ID: 1P-0-6TOP-HD-24-L-5Hx2W-0839

Number of Poles: 1

Dealer: _____

Date Sold: _____



Array Dimensions N/S	16.88 ft
Array Dimensions E/W	13.92 ft
Winter Tilt Angle	46
Front Edge Clearance	6 ft

MT Solar Bill of Materials (1P-0-6TOP-HD-24-L-5Hx2W-0839)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-24	24IN HD Wing	4
MTS-CLAMP-ANGLE-4PK	Angle Clamp	2

Rail Bill of Materials

Part	Qty
Rails (200in)	4
Rail Attachment	16
Module Mid Clamp	16
Module End Clamp	8
Ground Lug	2

Site Details:



Site Address: Crested Butte, CO 81224, USA

Array Specification

Duty Classification:	HD
Module Width:	40.00 in
Module Length:	82.50in
Number of Rows:	5
Number of Columns:	2
Total Number of Modules:	10
Winter Tilt Angle:	46
Front Edge Clearance:	6
Total Array Height at Tilt:	18.06 ft
Total Frame Length:	11.50 ft
Frame Weight:	860 lbs
Array Dimensions N/S:	16.88 ft
Array Dimensions E/W:	13.92 ft
Rail Length:	202.50 in
Rail Spacing:	3.44 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.07 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft
Foundation Volume:	3.025 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Crested Butte, CO 81224, USA
Wind Speed:	99 mph
Snow Load:	185 psf

Design Disclaimer

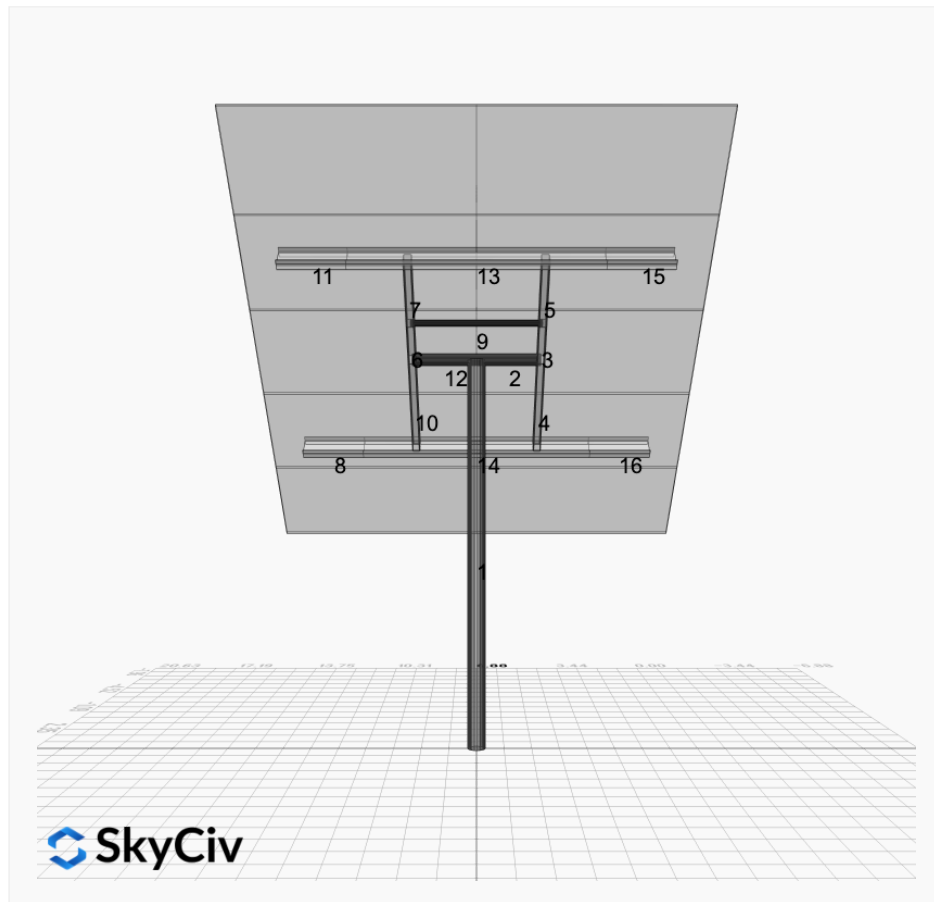
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

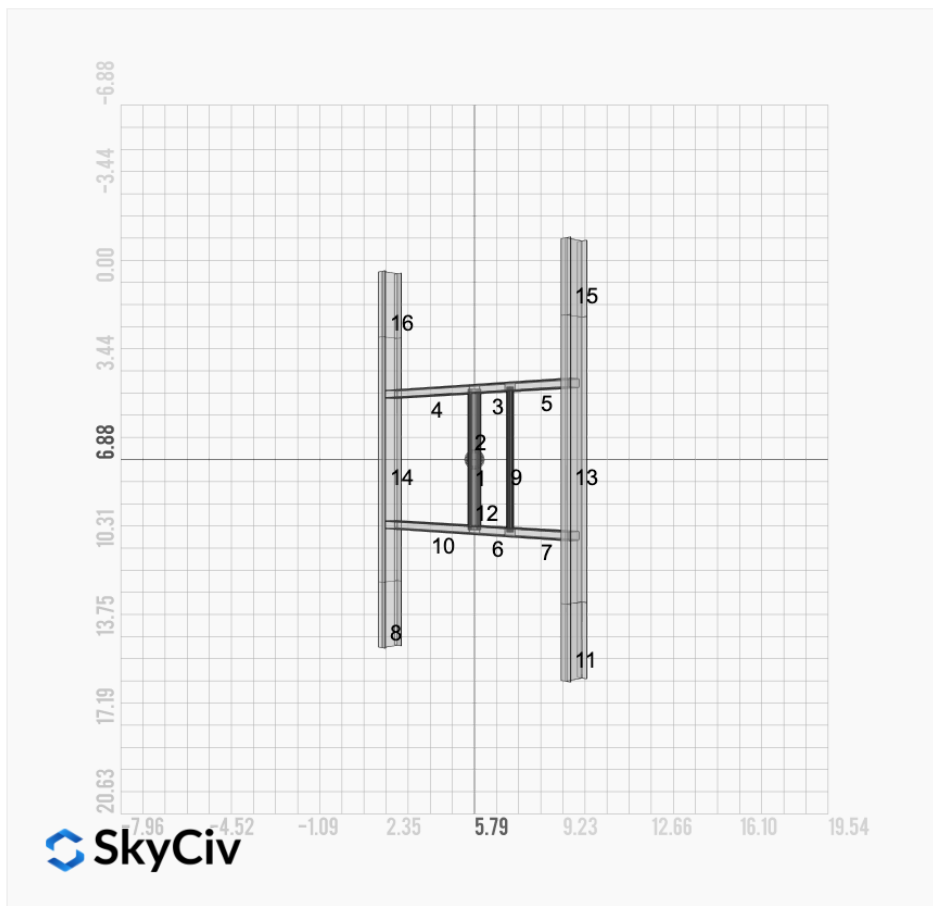
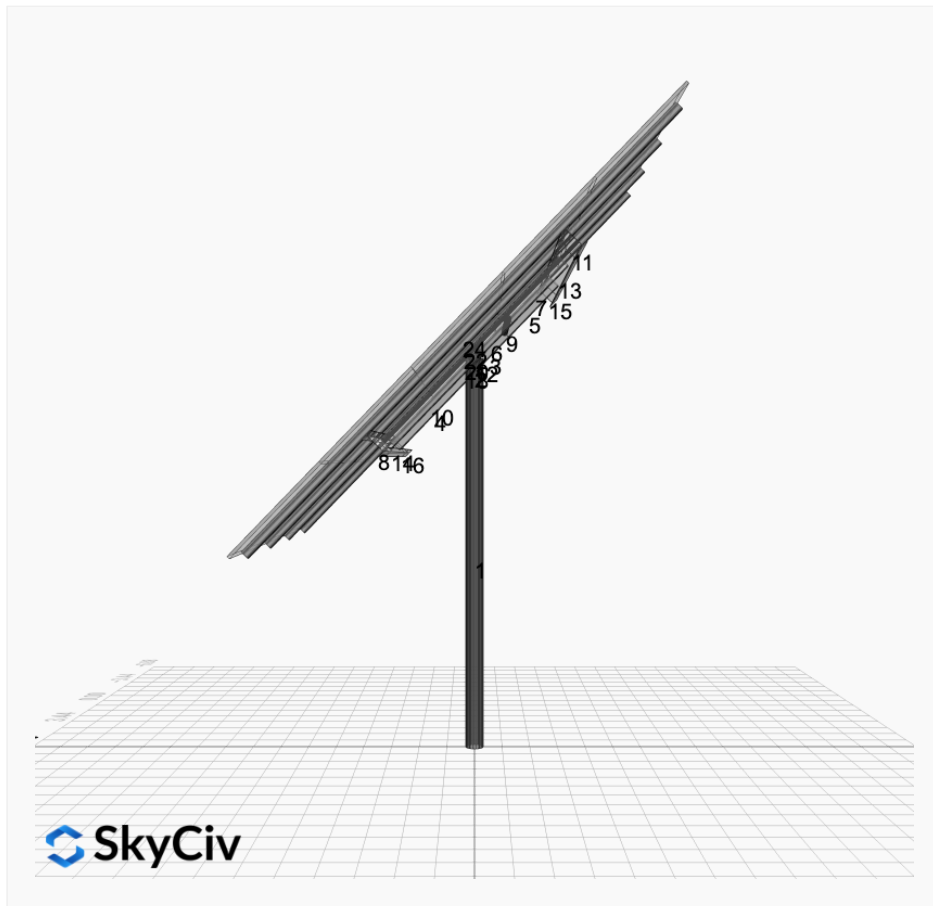
AutoDesigner Input

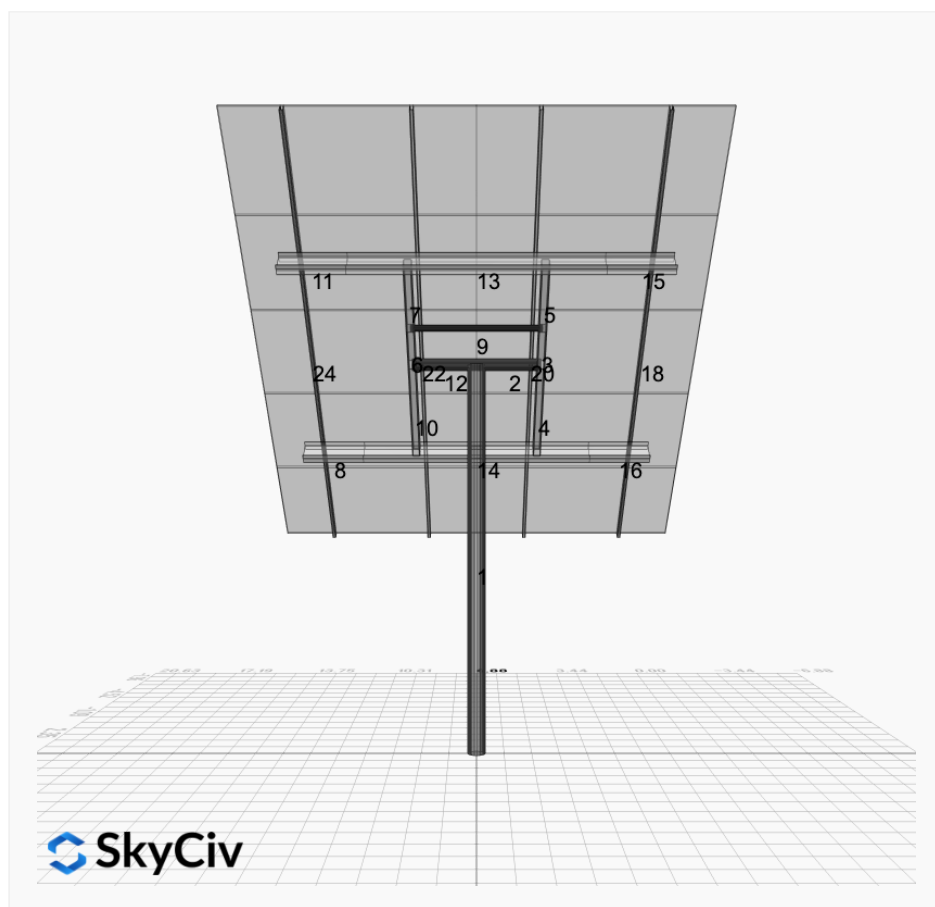
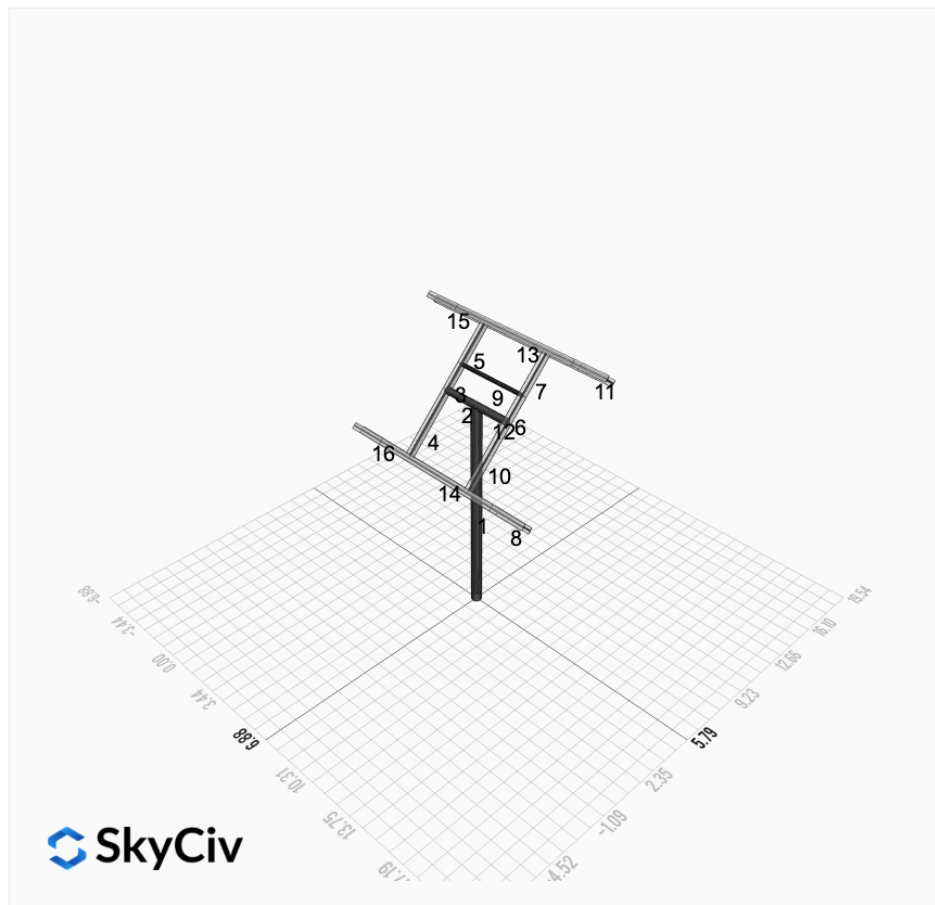
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Design Notes:

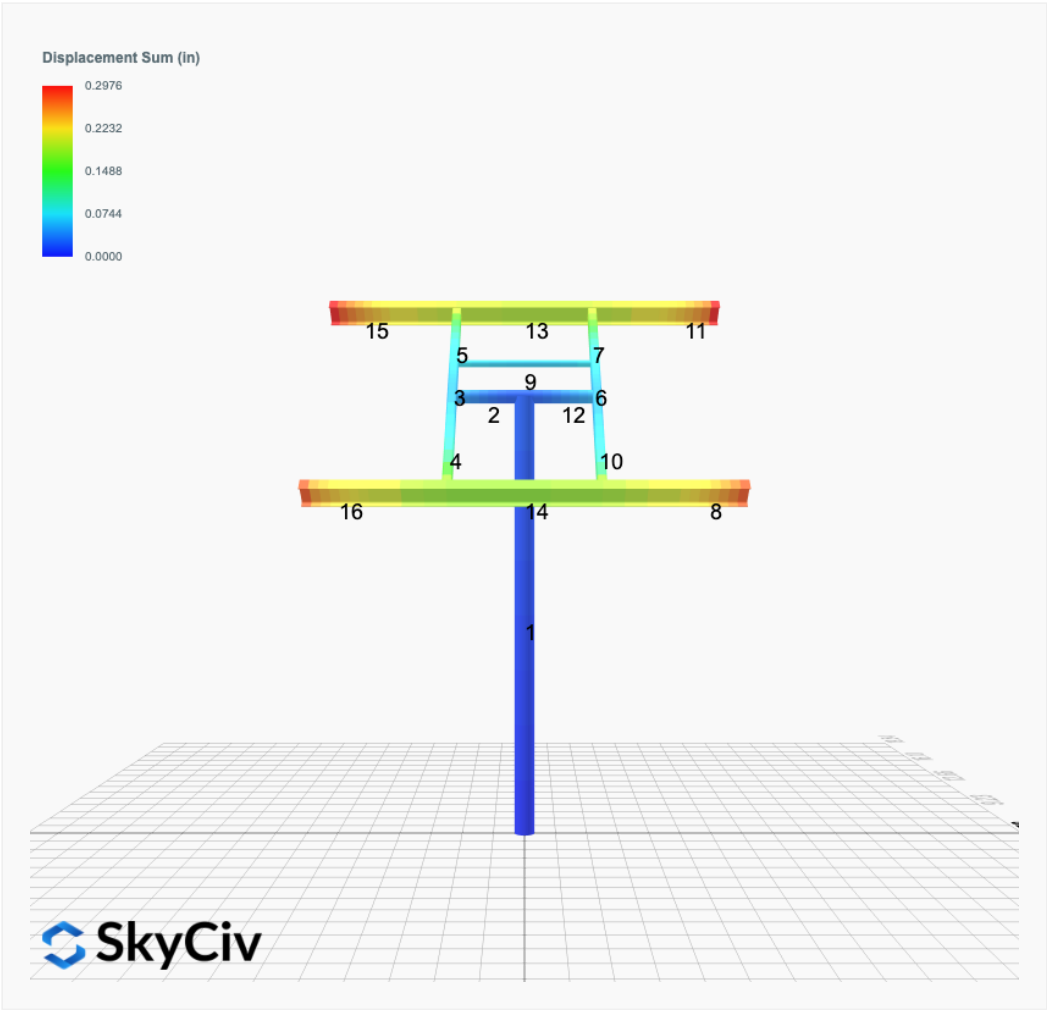
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



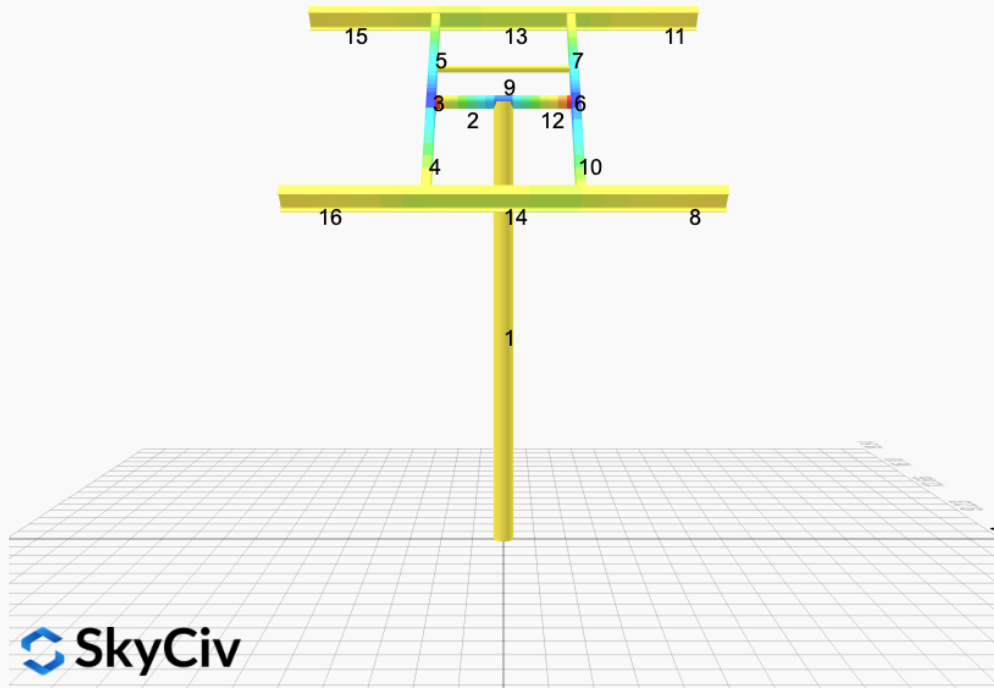




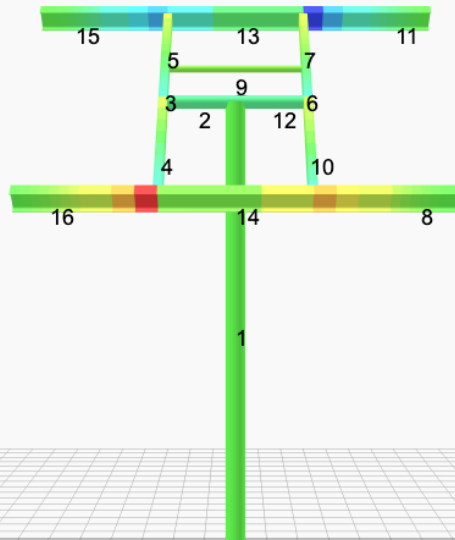
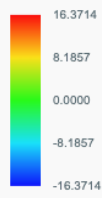
FEM Results (Envelope Worst Case for each member)



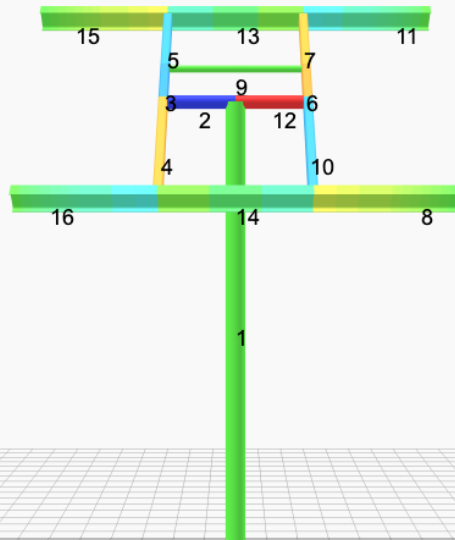
Top Bending Stress Z (ksi)



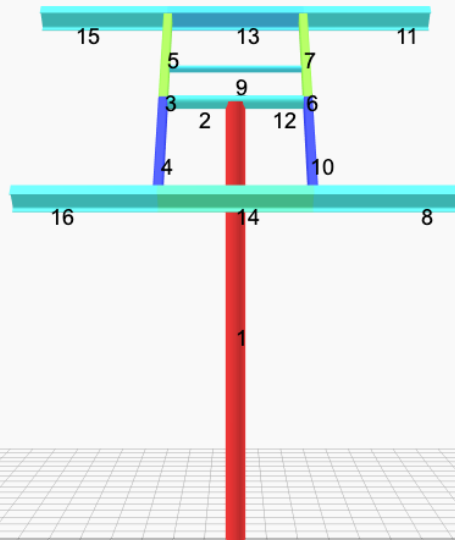
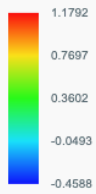
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 2. D + L	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 3. D + (S or Lr or R)	0.0000	8.3138	0.0000	0.0000	-0.0000	0.0819
ULS: 3. D + (S or Lr or R)	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.6684	0.0000	0.0000	-0.0000	0.0668
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 5b. D + 0.7E	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.6684	0.0000	0.0000	-0.0000	0.0668
ULS: 8. 0.6D + 0.7E	0.0000	1.0392	0.0000	0.0000	-0.0000	0.0129
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.9574	2.6566	0.0000	0.0000	-0.0000	11.7319
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.9574	0.8075	0.0000	0.0000	-0.0000	-11.3837
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7181	7.3618	0.0000	0.0000	-0.0000	8.8496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	6.6684	0.0000	0.0000	-0.0000	0.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7181	5.9750	0.0000	0.0000	-0.0000	-8.4871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	6.6684	0.0000	0.0000	-0.0000	0.0668
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7181	2.4255	0.0000	0.0000	-0.0000	8.8043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7181	1.0386	0.0000	0.0000	-0.0000	-8.5324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.7320	0.0000	0.0000	-0.0000	0.0215
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.9574	1.9638	0.0000	0.0000	-0.0000	11.7233
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0392	0.0000	0.0000	-0.0000	0.0129
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.9574	0.1147	0.0000	0.0000	-0.0000	-11.3923
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0392	0.0000	0.0000	-0.0000	0.0129

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	13.3798
Shear X	-1.5957
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	20.5696

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	8.3138
Shear X	-0.9574
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	11.7319

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Cabven - 5x2 - 48round - V1Jb
 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					
7	6in Pipe Sch 40	6.63	0.28					
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		
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_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

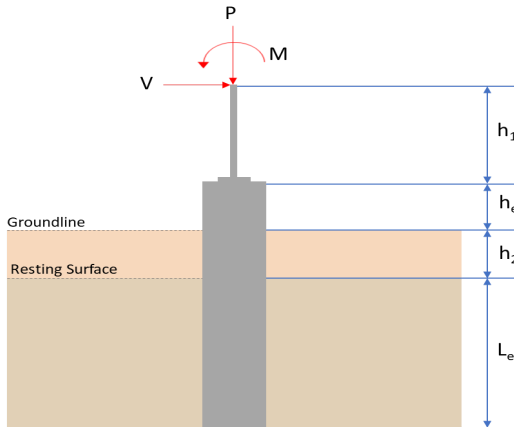
14	133.20	85.85	24.82	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62

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	0.195	0.486	0.000	0.021	0.000	0.537	#13	0.677	Not Required	Pass
2	0.001	0.449	0.103	0.110	0.013	0.553	#21	0.053	Not Required	Pass
3	0.019	0.578	0.156	0.057	0.046	0.744	#21	0.045	Not Required	Pass
4	0.019	0.571	0.163	0.057	0.039	0.724	#21	0.080	Not Required	Pass
5	0.019	0.359	0.162	0.057	0.038	0.392	#21	0.074	Not Required	Pass
6	0.019	0.578	0.156	0.057	0.046	0.744	#21	0.045	Not Required	Pass
7	0.019	0.359	0.162	0.057	0.038	0.392	#21	0.074	Not Required	Pass
8	0.000	0.025	0.123	0.021	0.017	0.149	#21	Not Required	Not Required	Pass
9	0.003	0.030	0.050	0.001	0.000	0.081	#21	0.136	Not Required	Pass
10	0.019	0.571	0.163	0.057	0.039	0.724	#21	0.080	Not Required	Pass
11	0.000	0.025	0.123	0.021	0.017	0.149	#21	Not Required	Not Required	Pass
12	0.001	0.449	0.103	0.110	0.013	0.553	#21	0.053	Not Required	Pass
13	0.007	0.134	0.434	0.038	0.032	0.551	#21	0.190	Not Required	Pass
14	0.011	0.138	0.434	0.038	0.032	0.551	#21	0.190	Not Required	Pass
15	0.000	0.025	0.123	0.021	0.017	0.149	#21	Not Required	Not Required	Pass
16	0.000	0.025	0.123	0.021	0.017	0.149	#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																										
	<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>Pile Input</div> <div></div> <div>Geometry</div> <div>Pile shape: round</div> <div>D = 48 in - Pile diameter</div> <div>L = 6.5 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>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>Tabulation of Loads</div> <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>8.314</td><td>13.380</td></tr><tr><td>Vx (kip)</td><td>-0.957</td><td>-1.596</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>11.732</td><td>20.570</td></tr></table> <div>Material Properties</div> <div>f'ck = 2.5 ksi - Concrete strength,</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)	8.314	13.380	Vx (kip)	-0.957	-1.596	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	11.732	20.570	
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Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	0.000																										
Mz (kipft)	11.732	20.570																										
	<div>Required depth to resist lateral loads (ASD)</div> <div>H - Point of application of the lateral load</div> <div>H = h1 + h2 + he</div> <div>H = (0 ft) + (0 ft) + (0 ft)</div> <div>H = 0 ft</div> <div>Considering x-direction:</div> <div>Ho - Lateral force per length of pile,</div> <div>Ho = Vz / D</div> <div>Ho = (-0.957 kip) / (48 in)</div> <div>Ho = -0.23925 kip/ft</div> <div>Mo - Moment per length of pile,</div>																											

	$M_o = \frac{M_z + (V_x H)}{D}$ $M_o = \frac{(11.732 \text{ kipft}) + ((-0.957 \text{ kip}) \times (0 \text{ ft}))}{(48 \text{ in})}$ $M_o = 2.933 \text{ kipft/ft}$ <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: $L_{e,x} = 6.1299 \text{ ft}$ - 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> $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(6.1299 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.13 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ <p>Ratio - Embedded depth</p> $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(6.13 \text{ ft})}{(6.5 \text{ ft})}$ $Ratio = 0.94308$	<p>Status: PASS Ratio: 0.940</p>
	<p>End-bearing Capacity (ASD) A - Pile cross-section area</p> $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(48 \text{ in})}{2}\right)^2$ $A = 12.566 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_v}{A}$ $q = \frac{(8.314 \text{ kip})}{(12.566 \text{ ft}^2)}$ $q = 0.66161 \text{ kip/ft}^2$ <p>Check bearing capacity ratio: Ratio - Capacity</p> $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.66161 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.3308$	<p>Status: PASS Ratio: 0.330</p>
Czeraniak	Lateral Soil Pressure (ASD):	

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.23925 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.933 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.933 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.23925 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (2.933 \text{ kipft/ft})) + (4 \times (-0.23925 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4748 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.933 \text{ kipft/ft})) + (3 \times (-0.23925 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (2.933 \text{ kipft/ft})) + (2 \times (-0.23925 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.24475 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.933 \text{ kipft/ft})) + ((-0.23925 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.96166 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4748 \text{ ft})}{2}$$

$$p_a = 0.33561 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24475 \text{ kip/ft}^2)}{(0.33561 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.72927$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

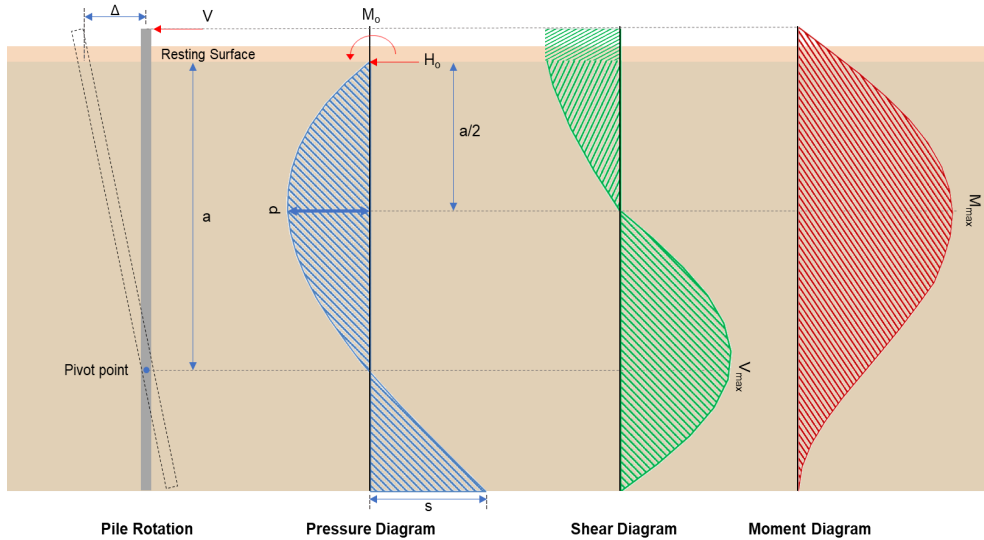
$$= \frac{(0.96166 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.730**

$$Ratio = \frac{\dots}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.98631$$

Status: **PASS**
Ratio: **0.990**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.596 \text{ kip})}{(48 \text{ in})}$$

$$H_o = -0.399 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.57 \text{ kipft}) + ((-1.596 \text{ kip}) \times (0 \text{ ft}))}{(48 \text{ in})}$$

$$M_o = 5.1425 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.1425 \text{ kipft/ft})}{(-0.399 \text{ kip/ft})}$$

$$E = 12.888 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1425 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.399 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (5.1425 \text{ kipft/ft})) + (4 \times (-0.399 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4696 \text{ ft}$$

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

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

$$V_{max} = ((-0.399 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.888 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4696 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.888 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4696 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 6.6534 \text{ kip}$ <p>M_{max} - Max bending moment located at depth $a/2$,</p> $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.399 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(12.888 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4696 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.888 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4696 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (12.888 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4696 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 20.68 \text{ kipft}$	
<p>Table 22.4.2.1</p> <p>22.4.2.2, 10.6.1.1</p>	<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, $A_g = 1809.6 \text{ in}^2$ - Gross area of concrete,</p> <p>Longitudinal reinforcement:</p> <p>Required reinforcement due to axial load, $A_{st,required}$</p> $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(13.38 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1809.6 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1809.6 \text{ in}^2)) \right]$ $A_{st,required} = -66.023 \text{ in}^2$ <p>A_{min} - Governing minimum reinforcement area,</p> $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-66.023 \text{ in}^2), (0.0018 \times (1809.6 \text{ in}^2))]$ $A_{min} = 3.2572 \text{ in}^2$ <p>n_{rebar} - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(3.2572 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 12$ <p>A_{st} - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (12) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 3.6816 \text{ in}^2$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(3.2572 \text{ in}^2)}{(3.6816 \text{ in}^2)}$ $\text{Ratio} = 0.88474$ <p>25.2.3 s_{rebar} - Minimum spacing of reinforcement,</p>	<p>Status: PASS Ratio: 0.880</p>

	<div><div>$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}$</div><div>Ties: Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in) s_{ties} - Maximum center-to-center spacing of ties, $s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), D]$$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (48 \text{ in})]$$s_{ties} = 10 \text{ in}$</div><div>Summary: Main reinforcement: 12 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</div></div>	
22.4.2.2	<div><div>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})]$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1809.6 \text{ in}^2) - (3.6816 \text{ in}^2)]) + ((60 \text{ ksi}) \times (3.6816 \text{ in}^2))]$$\phi P_N = 2242.3 \text{ kip}$ $Ratio$ - Capacity $Ratio = \frac{P}{\phi P_N}$$Ratio = \frac{(13.38 \text{ kip})}{(2242.3 \text{ kip})}$$Ratio = 0.0059672$</div></div>	Status: PASS Ratio: 0.010
22.5.2.2	<div><div>Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$$d = 0.80 \times (48 \text{ in})$$d = 38.4 \text{ in}$</div></div>	
22.5.5.1.3	<div><div>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$</div></div>	
22.5.5.1.1	<div><div>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}$</div></div>	
22.5.5.1.1(a)	<div><div>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.38 \text{ kip} \rightarrow 13380 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_o} \right] b_w d$</div></div>	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13380 \text{ lbf})}{6 \times (1809.6 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.76 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.76 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.76 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.76 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.57 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6534 \text{ kip}$ - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(6.6534 \text{ kip})}{(111.57 \text{ kip})}$$

$$Ratio = 0.059633$$

Status: **PASS**
 Ratio: **0.059633**

		Ratio: 0.000
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (48 \text{ in})^3}{32}$ $S_m = 10857 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 10857.344 \text{ in}^3$ $\phi M_{n,1} = 147.027 \text{ kipft}$ <p>14.5.2.1b $\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (10857 \text{ in}^3)$ $\phi M_{n,2} = 1249.7 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(147.03 \text{ kipft}), (1249.7 \text{ kipft})]$ $\phi M_n = 147.03 \text{ kipft}$ <p>Considering x-direction:</p> <p>$M_{max} = 20.68 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(20.68 \text{ kipft})}{(147.03 \text{ kipft})}$ $\text{Ratio} = 0.14066$	
		Status: PASS Ratio: 0.140