

Your Project Calculations



Project Name: MTSOLAR_LDA0E0H4C2KD

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_LDA0E0H4C2KD&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=7RlflfSoTvnwC0O5TzKirsTnlk2ki7EBsEggiAwwFwfGmhyKkiTQgEyelV2QUInvd

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-HD-84-L-4Hx3W-79F7
Duty Classification:	HD
Module Width:	44.63 in
Module Length:	90.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	30
Front Edge Clearance:	5
Total Array Height at Tilt:	12.48 ft
Total Frame Length:	21.50 ft
Frame Weight:	1147 lbs
Array Dimensions N/S:	15.04 ft
Array Dimensions E/W:	22.75 ft
Rail Length:	180.50 in
Rail Spacing:	3.75 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.76 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: 6.50 ft
Foundation Volume:	3.852 y ³
Foundation Result:	PASSED
Mount Twist:	0.000003 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Hico, TX 76457, USA
Wind Speed:	100 mph
Snow Load:	5 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.002199 ksf



Design Disclaimer

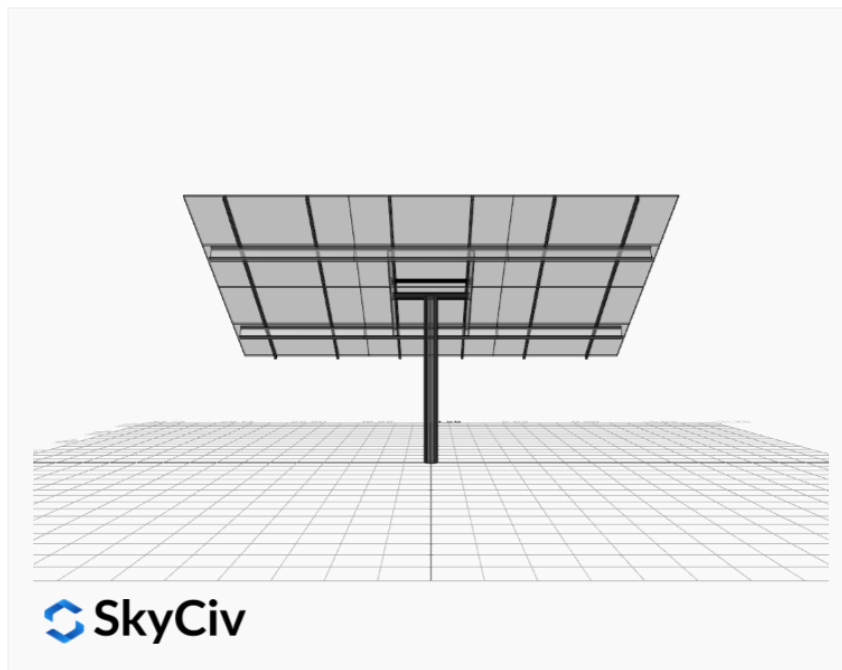
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

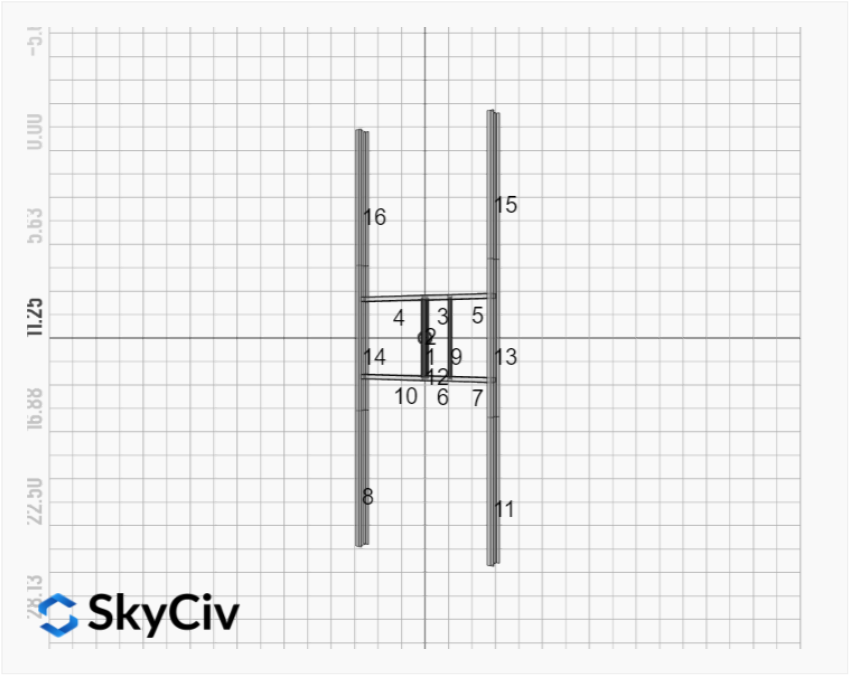
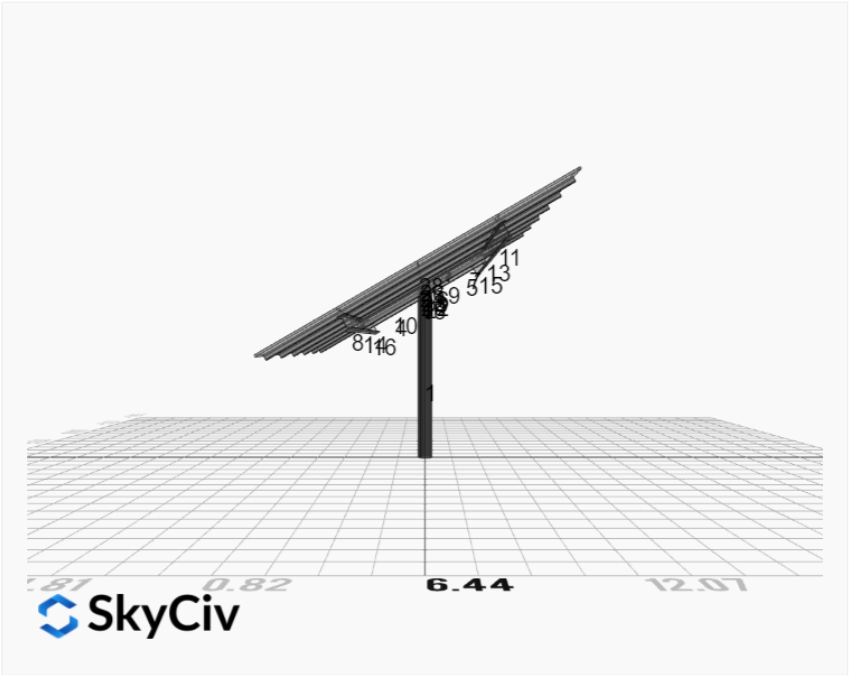
AutoDesigner Input

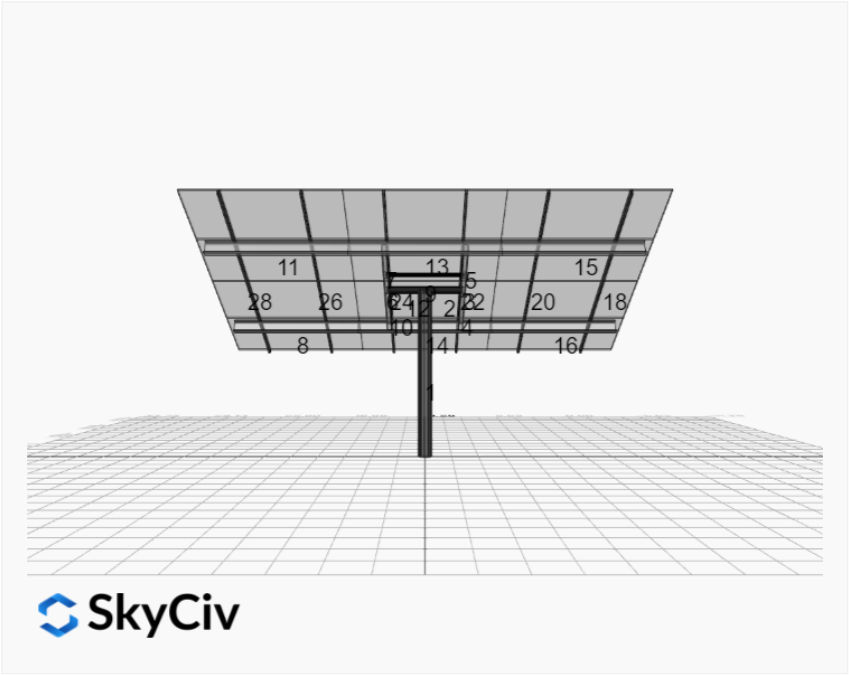
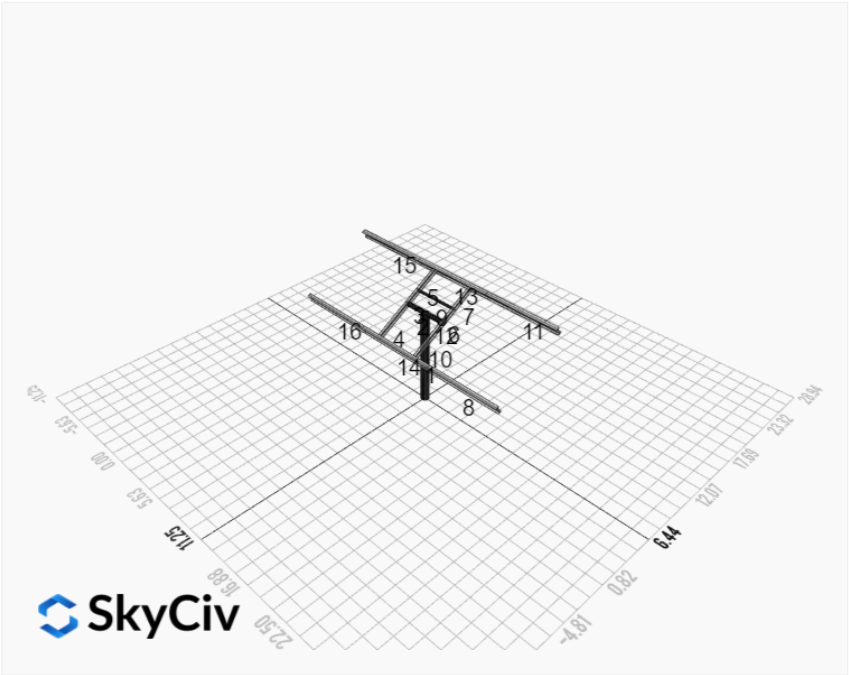
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Design Notes:

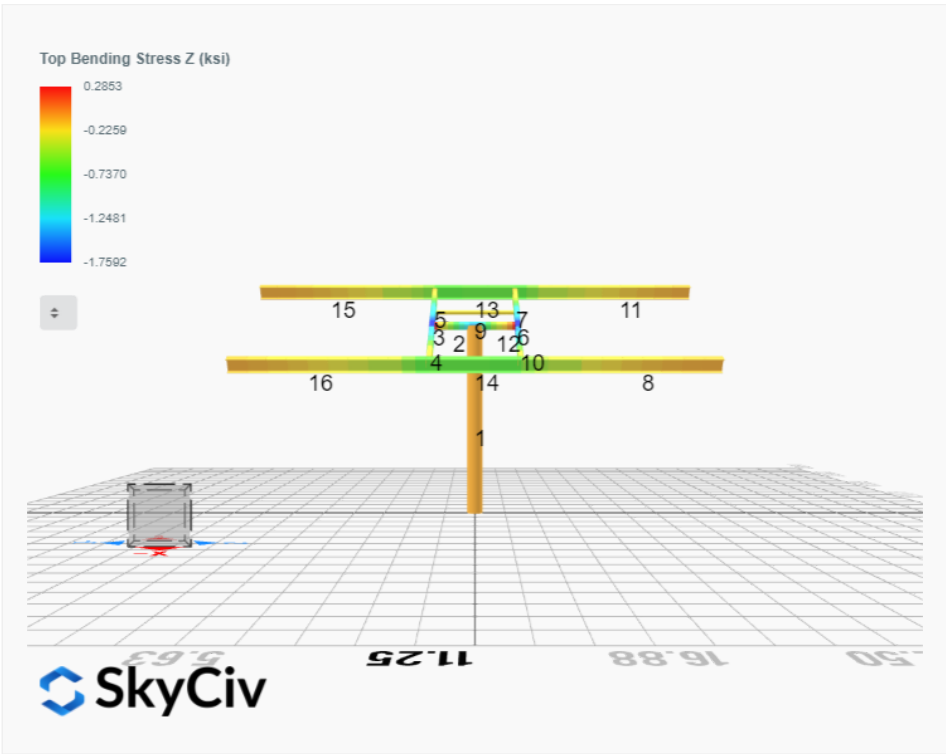
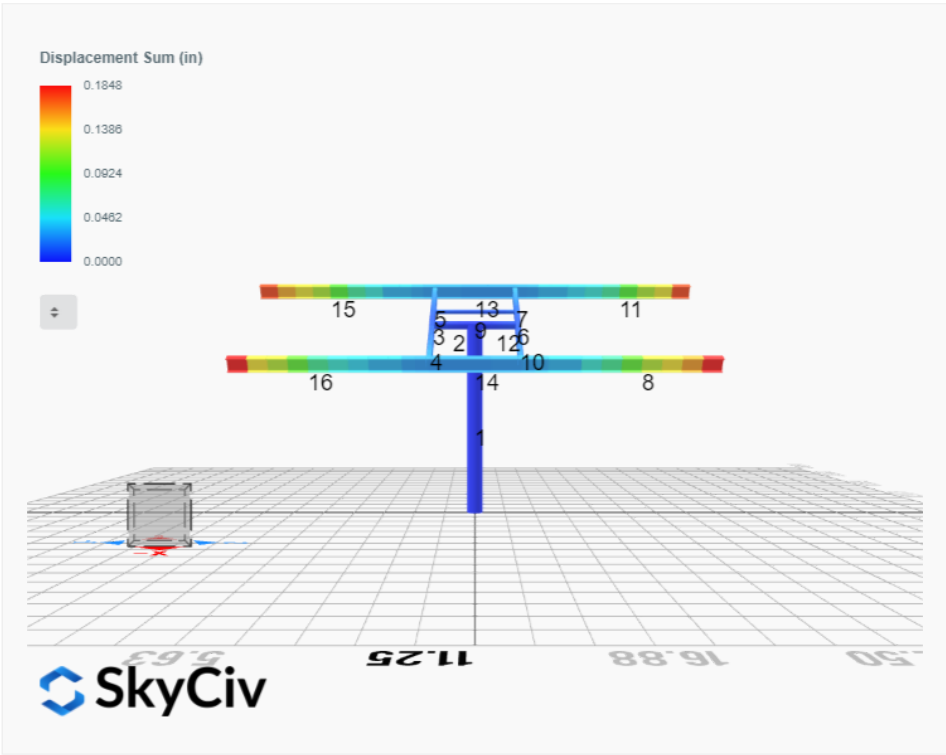
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

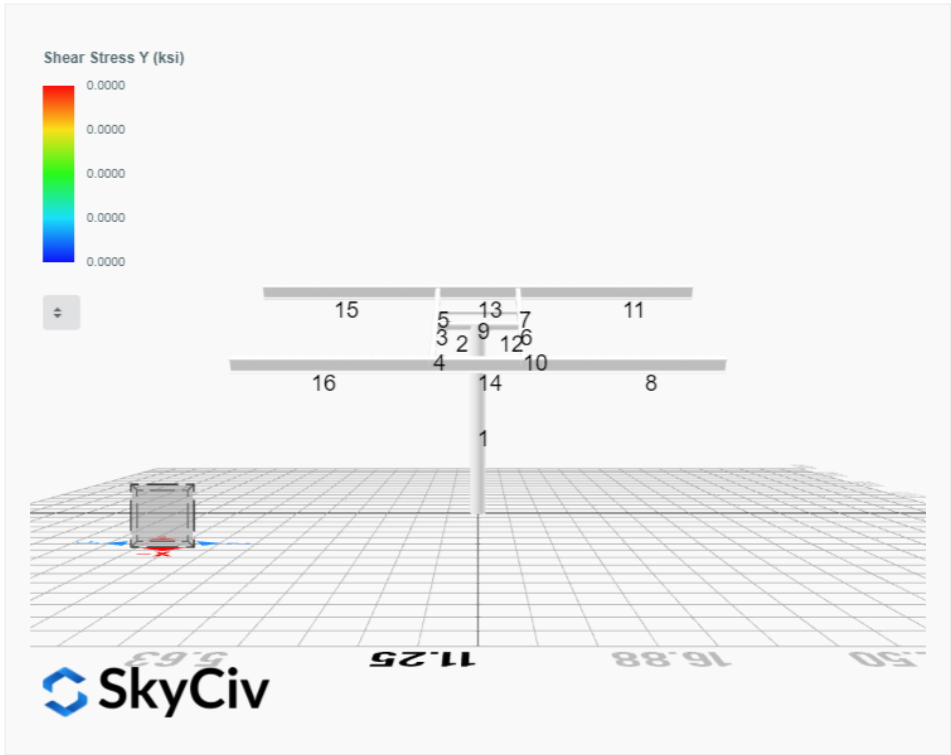
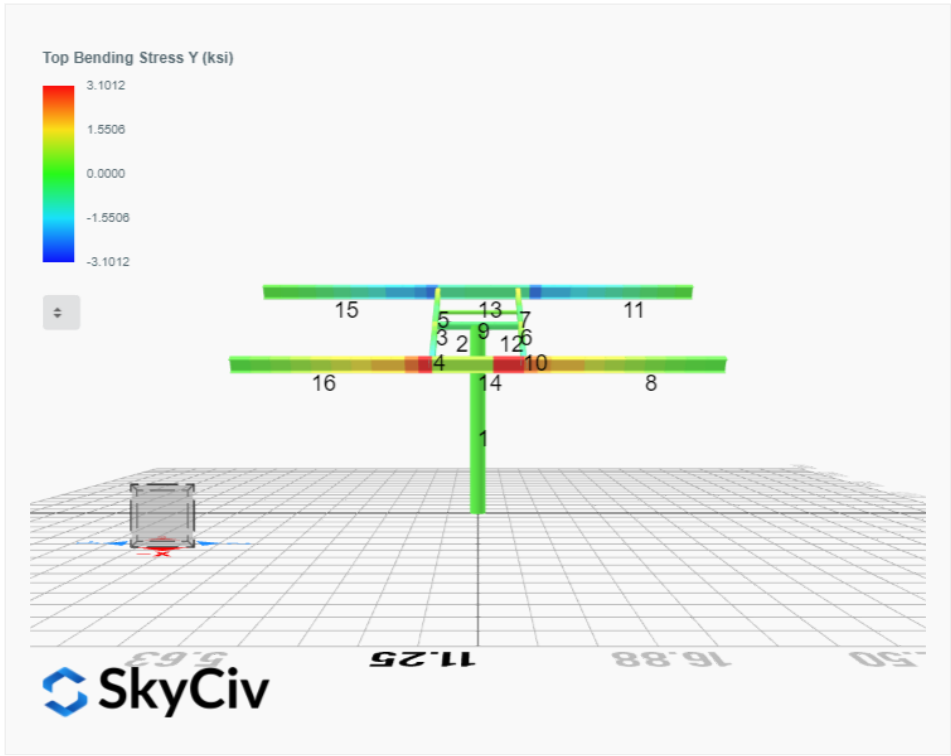


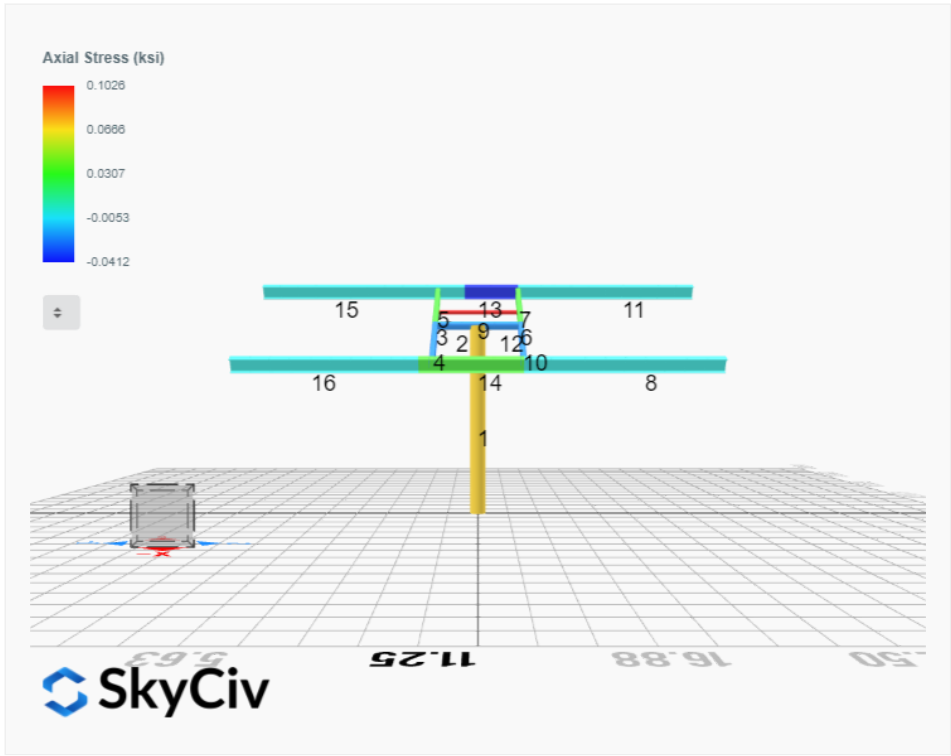




FEM Results (Envelope Worst Case for each member)







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	2.4441	0.0000	0.0000	-0.0000	0.0277
ULS: 2. D + L	-0.0000	2.4441	0.0000	0.0000	-0.0000	0.0277
ULS: 3. D + (S or Lr or R)	-0.0000	3.0600	0.0000	0.0000	-0.0000	0.0281
ULS: 3. D + (S or Lr or R)	-0.0000	2.4441	0.0000	0.0000	-0.0000	0.0277
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.9060	0.0000	0.0000	-0.0000	0.0280
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.4441	0.0000	0.0000	-0.0000	0.0277
ULS: 5b. D + 0.7E	-0.0000	2.4441	0.0000	0.0000	-0.0000	0.0277
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.9060	0.0000	0.0000	-0.0000	0.0280
ULS: 8. 0.6D + 0.7E	-0.0000	1.4664	0.0000	0.0000	-0.0000	0.0166
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2619	8.0938	0.0000	0.0000	-0.0000	29.0839
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.2619	8.0938	0.0000	0.0000	-0.0000	29.0839
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7959	-2.3985	-0.0000	0.0000	-0.0000	-24.1247
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.3299	-1.5914	-0.0000	0.0000	-0.0000	-31.5925
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4464	7.1433	0.0000	0.0000	-0.0000	21.8201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4464	7.1433	0.0000	0.0000	-0.0000	21.8201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0969	-0.7259	-0.0000	0.0000	-0.0000	-18.0863
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7474	-0.1206	0.0000	0.0000	-0.0000	-23.6871
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4464	6.6813	0.0000	0.0000	-0.0000	21.8198
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4464	6.6813	0.0000	0.0000	-0.0000	21.8198
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0969	-1.1879	-0.0000	0.0000	-0.0000	-18.0866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.7474	-0.5826	-0.0000	0.0000	-0.0000	-23.6874
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2619	7.1161	0.0000	0.0000	-0.0000	29.0728
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.2619	7.1161	0.0000	0.0000	-0.0000	29.0728
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7959	-3.3762	-0.0000	0.0000	-0.0000	-24.1358
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.3299	-2.5691	-0.0000	0.0000	-0.0000	-31.6036

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	12.6570
Shear X	-5.4364
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	53.0414

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	8.0938
Shear X	-3.2619
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	31.6036

Project Details

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

User Name: sales@mtsolar.us
 Project Name: MTSOLAR_LDA0E0H4C2KD
 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					
9	8in Pipe Sch 40	8.63	0.32					
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_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D	
1	9	18.40	18.40	8.76	-	300	200	1	
2	5	1.30	1.30	2.00	-	300	200	1	
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1	
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.54,1.67,1.67,1.66,1.61,1.67,1.67,1.65,1.69,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.64	300	200	1	
5	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67	300	200	1	
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1	
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67	300	200	1	
8	19	14.70	14.70	7.00	2.33,2.33	300	200	1	
9	2	2.60	2.60	4.00	-	300	200	1	
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.54,1.67,1.67,1.66,1.60,1.67,1.67,1.65,1.69,1.67,1.67,1.65,1.71,1.67,1.67,1.66,1.64	300	200	1	
11	19	14.70	14.70	7.00	2.33,2.33	300	200	1	
12	5	1.30	1.30	2.00	-	300	200	1	
13	19	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.02,1.02,1.02,1.03,1.02,1.02,1.02,1.05,1.02,1.02,1.02,1.04,1.02,1.02,1.02,1.03	300	200	1	
14	19	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.01,1.02,1.02,1.02,1.01,1.02,1.02,1.02,1.10,1.02,1.02,1.02,1.26,1.02,1.02,1.02,1.01	300	200	1	
15	19	14.70	14.70	7.00	2.33,2.33	300	200	1	
16	19	14.70	14.70	7.00	2.33,2.33	300	200	1	

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	250.11	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	15.17	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94

10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	15.17	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	23.37	6.12	40.24	43.62
14	133.20	104.94	23.14	6.12	40.24	43.62
15	133.20	15.17	32.87	6.12	40.24	43.62
16	133.20	15.17	32.87	6.12	40.24	43.62

Design Ratio

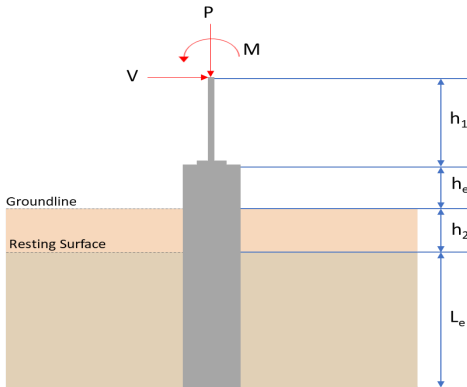
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.051	0.637	0.000	0.048	0.000	0.642	#32	0.376	Not Required	Pass
2	0.003	0.507	0.249	0.104	0.046	0.757	#13	0.035	Not Required	Pass
3	0.004	0.791	0.028	0.079	0.006	0.801	#13	0.045	Not Required	Pass
4	0.004	0.788	0.105	0.079	0.021	0.837	#13	0.080	Not Required	Pass
5	0.004	0.491	0.110	0.079	0.028	0.516	#13	0.074	Not Required	Pass
6	0.004	0.791	0.028	0.079	0.006	0.801	#13	0.045	Not Required	Pass
7	0.004	0.491	0.110	0.079	0.028	0.516	#13	0.074	Not Required	Pass
8	0.000	0.229	0.157	0.053	0.006	0.354	#13	Not Required	Not Required	Pass
9	0.014	0.089	0.056	0.001	0.000	0.151	#13	0.204	Not Required	Pass
10	0.004	0.788	0.105	0.079	0.021	0.837	#13	0.080	Not Required	Pass
11	0.000	0.229	0.157	0.053	0.006	0.354	#13	Not Required	Not Required	Pass
12	0.003	0.507	0.249	0.104	0.046	0.757	#13	0.035	Not Required	Pass
13	0.005	0.513	0.245	0.067	0.008	0.698	#13	0.190	Not Required	Pass
14	0.005	0.519	0.245	0.067	0.008	0.698	#13	0.190	Not Required	Pass
15	0.000	0.229	0.157	0.053	0.006	0.354	#13	Not Required	Not Required	Pass
16	0.000	0.229	0.157	0.053	0.006	0.354	#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

ON
NG

Capacity is provided
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 = 6.5 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>8.094</td><td>12.657</td></tr><tr><td>Vx (kip)</td><td>-3.262</td><td>-5.436</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>31.604</td><td>53.041</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)	8.094	12.657	Vx (kip)	-3.262	-5.436	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	31.604	53.041	
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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Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	0.000																										
Mz (kipft)	31.604	53.041																										
	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, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.262 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.51943 \text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(31.604 \text{ kipft}) + ((-3.262 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 5.0325 \text{ kipft/ft}$ 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: $L_{e,x} = 5.9972 \text{ ft}$ - 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, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.9972 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.997 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.5 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.997 \text{ ft})}{(6.5 \text{ ft})}$ $Ratio = 0.92262$	Status: PASS Ratio: 0.920
	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 $q = \frac{P_v}{A}$ $q = \frac{(8.094 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.50587 \text{ kip/ft}^2$ Check bearing capacity ratio: <i>Ratio</i> - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.50587 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.25294$	Status: PASS Ratio: 0.250
Czerniak	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:

$H_o = -0.51943$ kip/ft - Lateral force per length of pile,

$M_o = 5.0325$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0325 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.51943 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (5.0325 \text{ kipft/ft})) + (4 \times (-0.51943 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5007 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (5.0325 \text{ kipft/ft})) + (3 \times (-0.51943 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^3 [(3 \times (5.0325 \text{ kipft/ft})) + (2 \times (-0.51943 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.21277 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.0325 \text{ kipft/ft})) + ((-0.51943 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.94987 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5007 \text{ ft})}{2}$$

$$p_a = 0.33755 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21277 \text{ kip/ft}^2)}{(0.33755 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63033$$

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

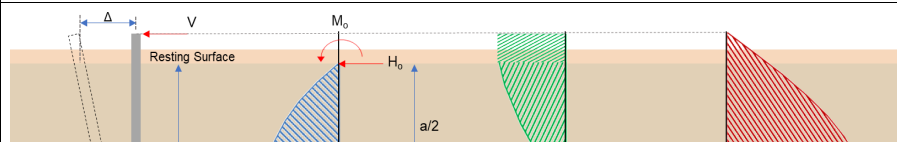
$$\text{Ratio} = \frac{s}{p_s}$$

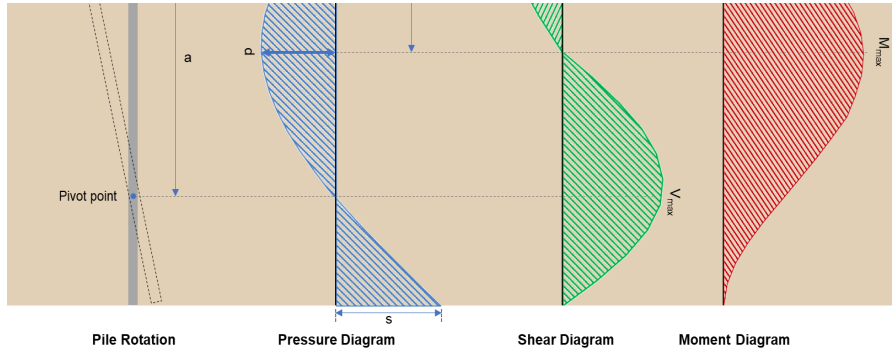
$$\text{Ratio} = \frac{(0.94987 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97423$$

Status: **PASS**
Ratio: **0.630**

Status: **PASS**
Ratio: **0.970**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.436 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.86561 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.041 \text{ kipft}) + ((-5.436 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.446 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.446 \text{ kipft/ft})}{(-0.86561 \text{ kip/ft})}$$

$$E = 9.7574 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.446 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.86561 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.446 \text{ kipft/ft})) + (4 \times (-0.86561 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5 \text{ ft}$$

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

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

$$V_{max} = ((-0.86561 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.7574 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.5 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.7574 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.5 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.48 \text{ kip}$$

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

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

$$M_{max} = ((-0.86561 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(9.7574 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.5 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.7574 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.5 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.7574 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.5 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 35.271 \text{ kipft}$	
		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, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(12.657 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.176 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.176 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\emptyset$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	
			Status: PASS Ratio: 0.970

	Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2)) \right]$ $\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(12.657 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0047313$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
22.5.5.1.1	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 $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.657 \text{ kip} \rightarrow 12657 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12657 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 120.17 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (120.17 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 120.17 \text{ kip}$	

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) $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.17 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.19 \text{ kip}$ Considering x-direction: $V_{max} = 11.48 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(11.48 \text{ kip})}{(111.19 \text{ kip})}$ $Ratio = 0.10324$	Status: PASS Ratio: 0.100
14.5.2.1b	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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = 0.85 f'_c S_m$	

	$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ 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: $M_{max} = 35.271 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(35.271 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.14131$	
		Status: PASS Ratio: 0.140