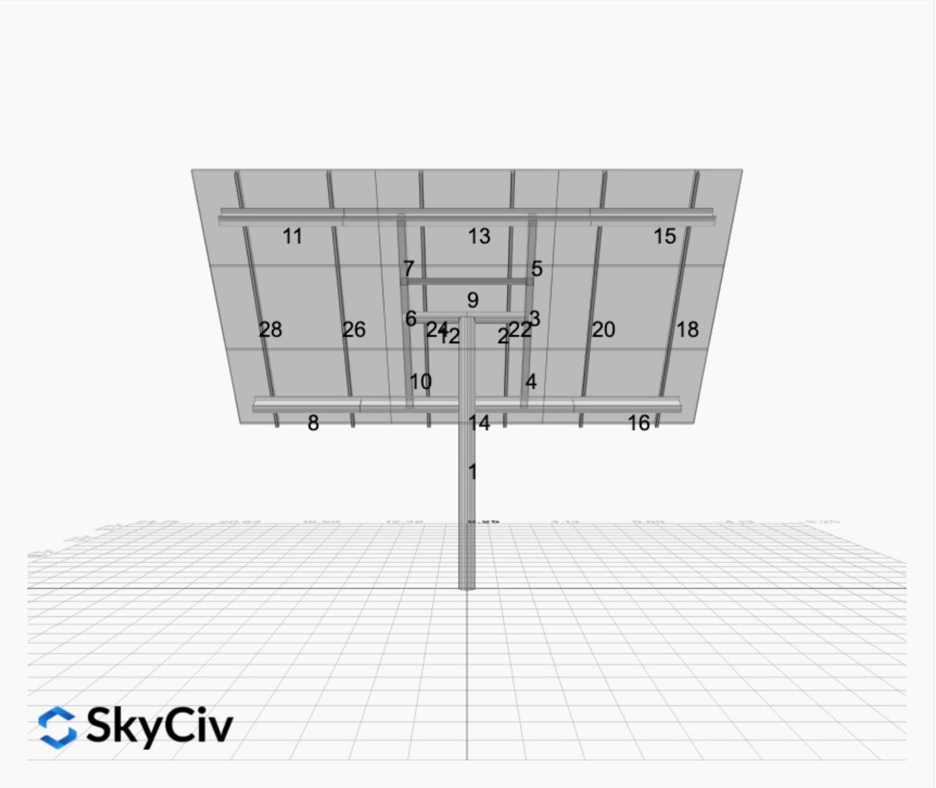
Project Details

Project Name: Woodard bay field array 3x3 - V1Jb
Location: 6336 Woodard Bay Rd NE, Olympia, WA 98506, USA
Unique ID: 1P-0-6TOP-SD-45-L-3Hx3W-F3EI
Dealer: ______________________________

Date: Mon Aug 04 2025
Number of Modules: 9
Number of Poles: 1
Date Sold:
____



Array Dimensions N/S	10.13 ft
Array Dimensions E/W	16.50 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

MT Solar Bill of Materials (1P-0-6TOP-SD-45-L-3Hx3W-F3EI)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-45	45IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (122in)	6
Rail Attachment	12
Module Mid Clamp	12
Module End Clamp	12
Ground Lug	3

Site Details:

Site Address: 6336 Woodard Bay Rd NE, Olympia, WA 98506, USA

Array Specification

Duty Classification:	SD
Module Width:	40.00 in
Module Length:	65.00in
Number of Rows:	3
Number of Columns:	3
Total Number of Modules:	9
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	12.76 ft
Total Frame Length:	15.00 ft
Module Info/Notes:	JAPAN SOLAR 300W
Array Dimensions N/S:	10.13 ft
Array Dimensions E/W:	16.50 ft
Rail Length:	121.50 in
Rail Spacing:	2.75 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.88 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: 5.00 ft
Foundation Volume:	2.963 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	6336 Woodard Bay Rd NE, Olympia, WA 98506, USA
Wind Speed:	92 mph
Snow Load:	15 psf

Design Disclaimer

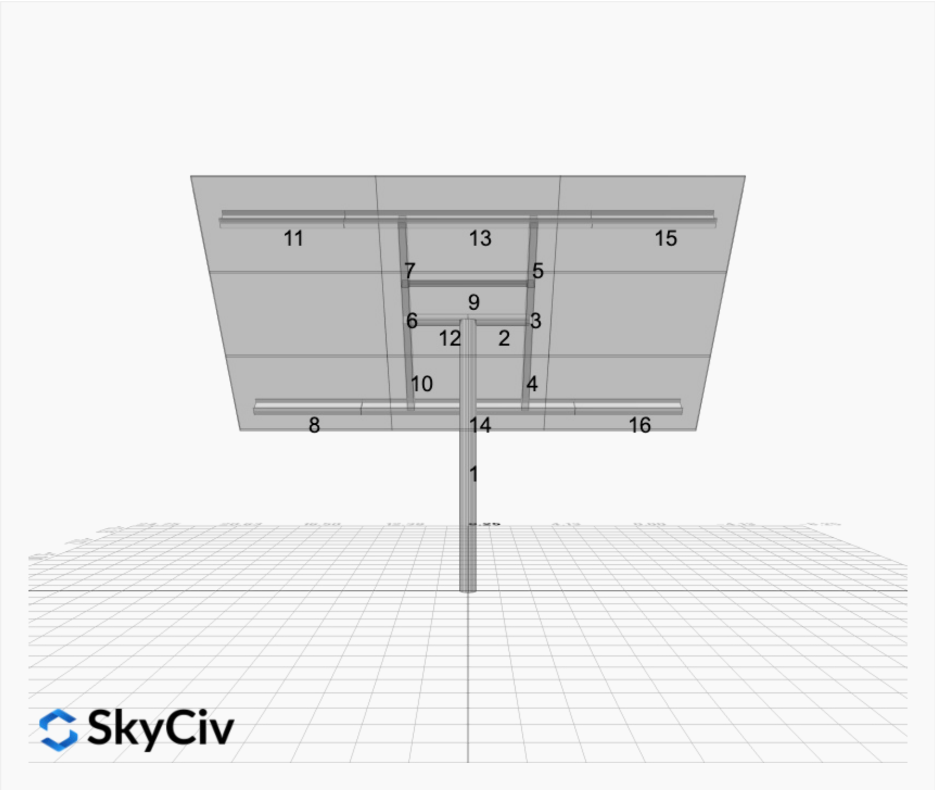
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

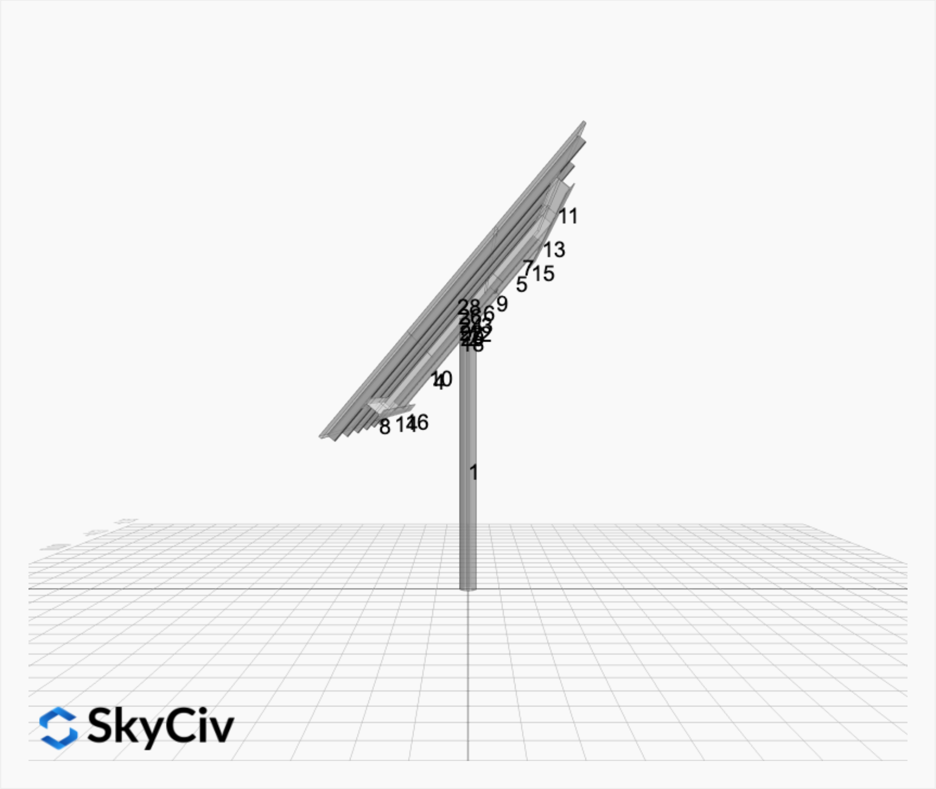
AutoDesigner Input

{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"","designer_email":"driftin_180sx@hotmail.com","designer_phone":"4257366454","project_id":"Woodard bay field array 3x3 - V1Jb","site_address":"6336 Woodard Bay Rd NE, Olympia, WA 98506, USA","module_info":"JAPAN SOLAR 300W","module_width":40,"module_length":65,"number_rows":"3","number_columns":3,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":50,"ground_clearance":5,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}

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)

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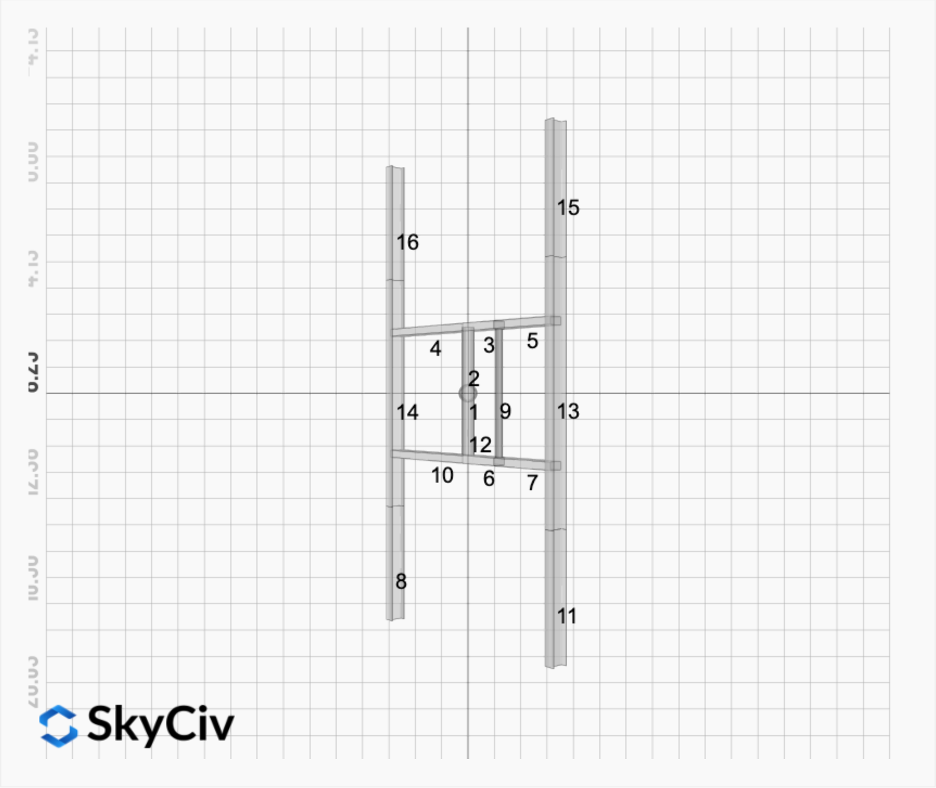
SkyCiv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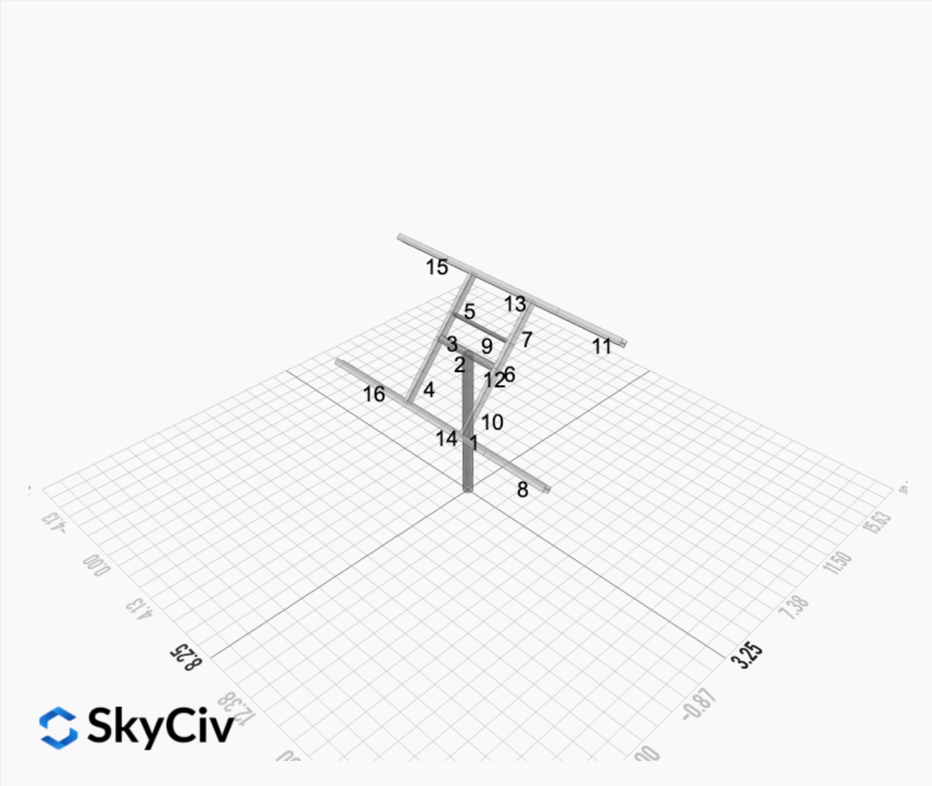
SkyCiv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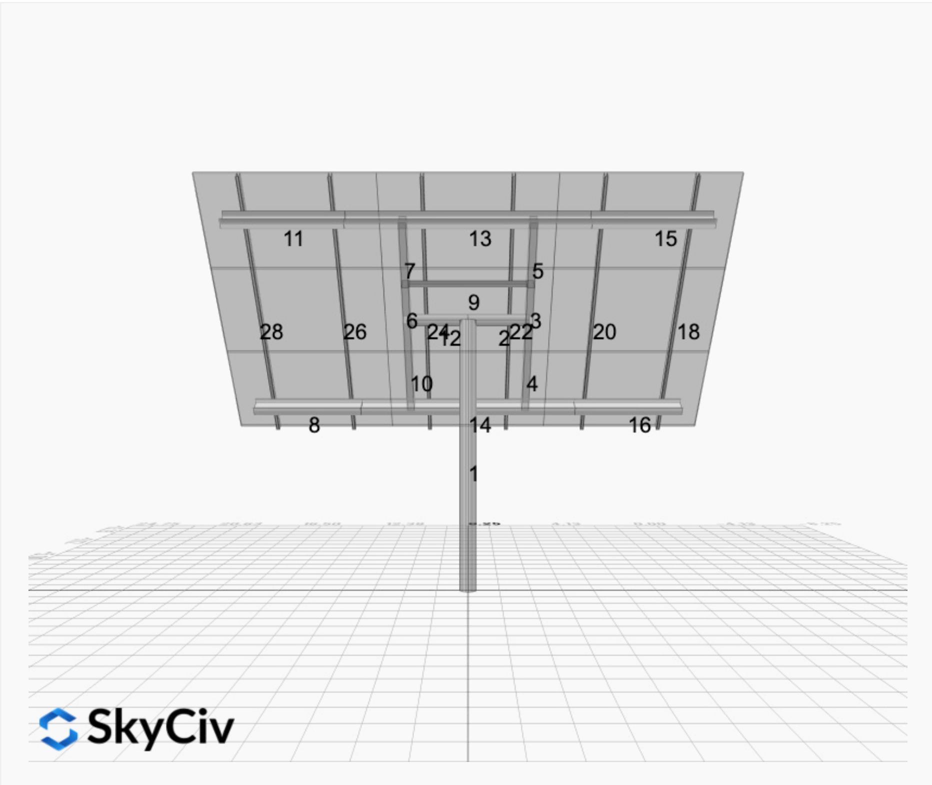
7.38
3.25
-0.87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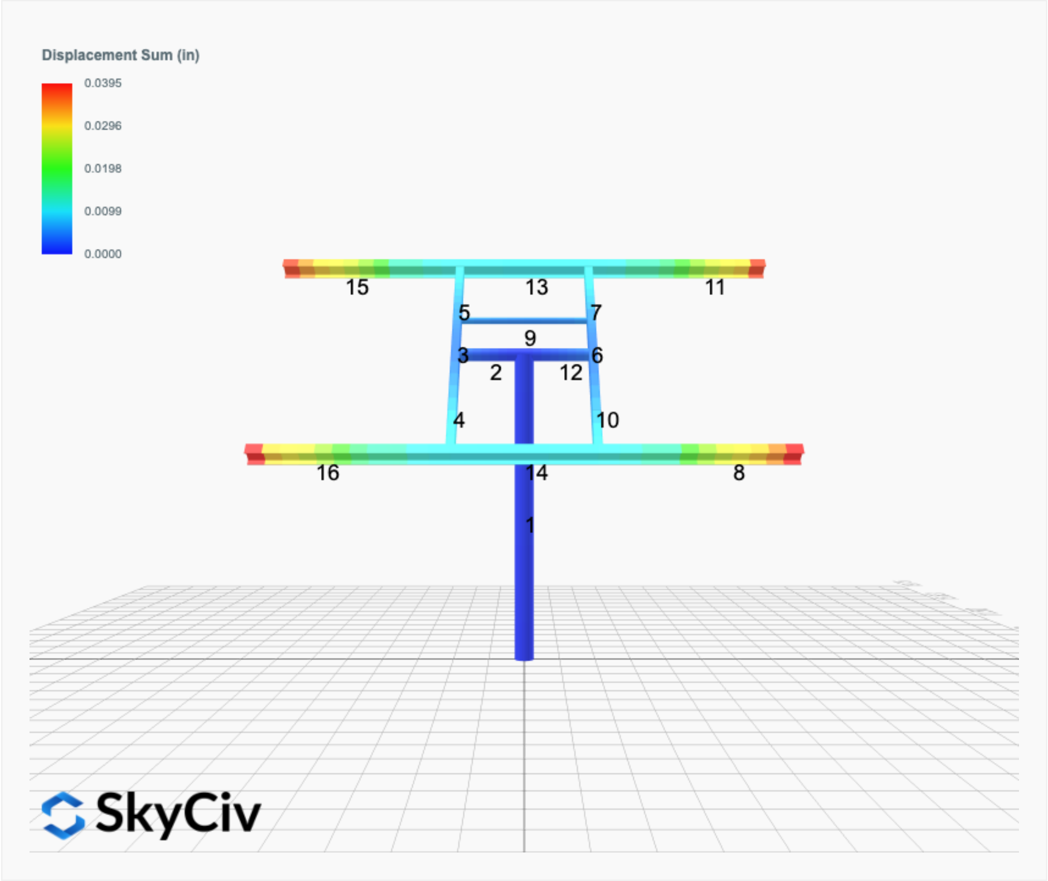
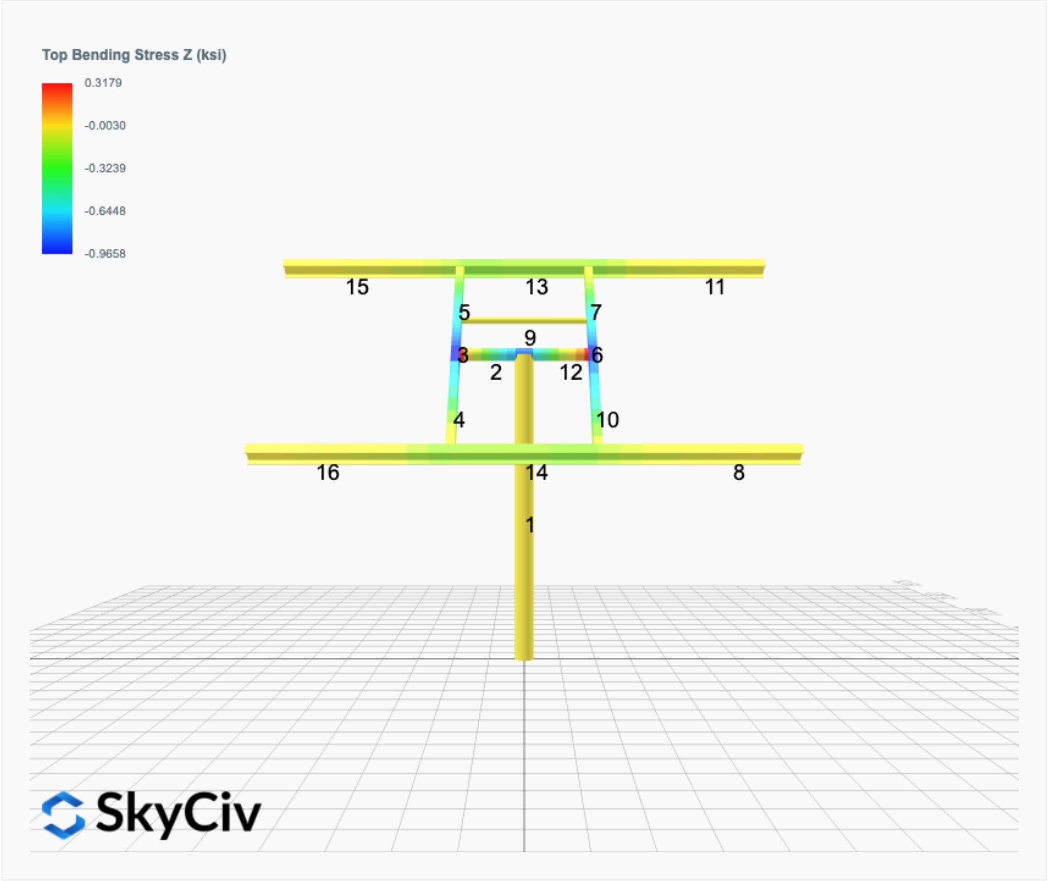
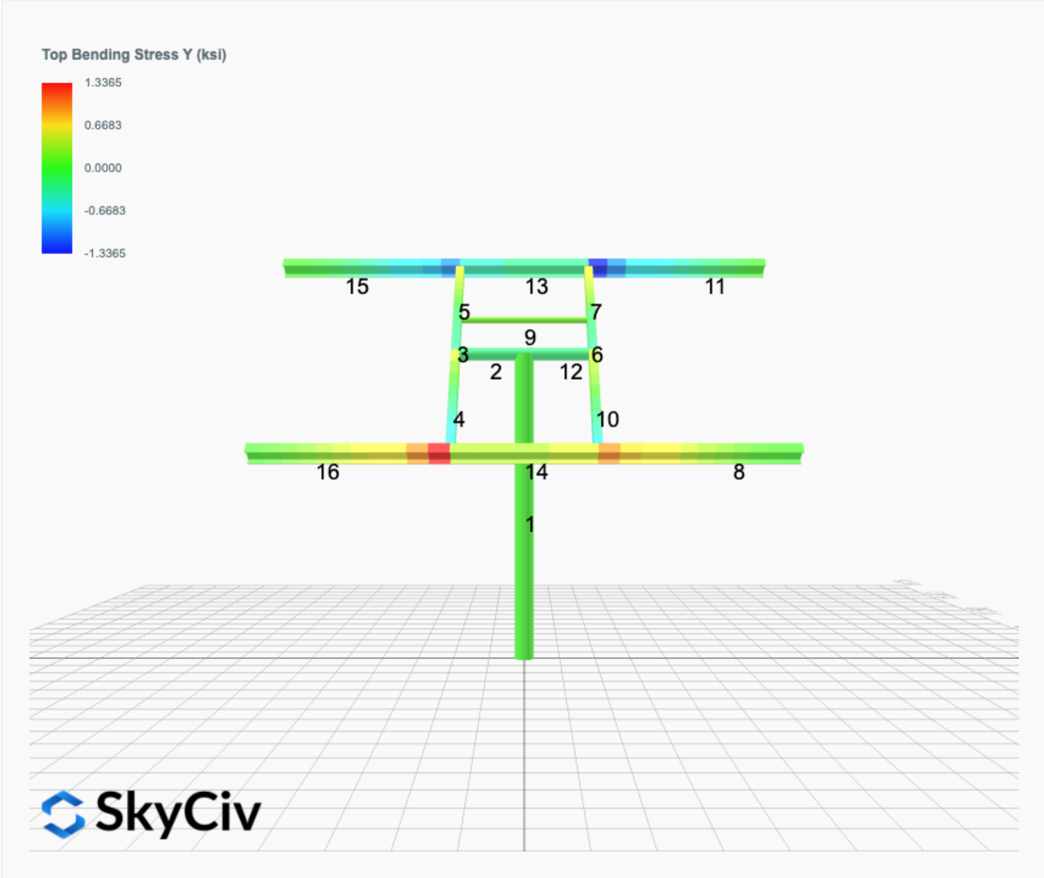
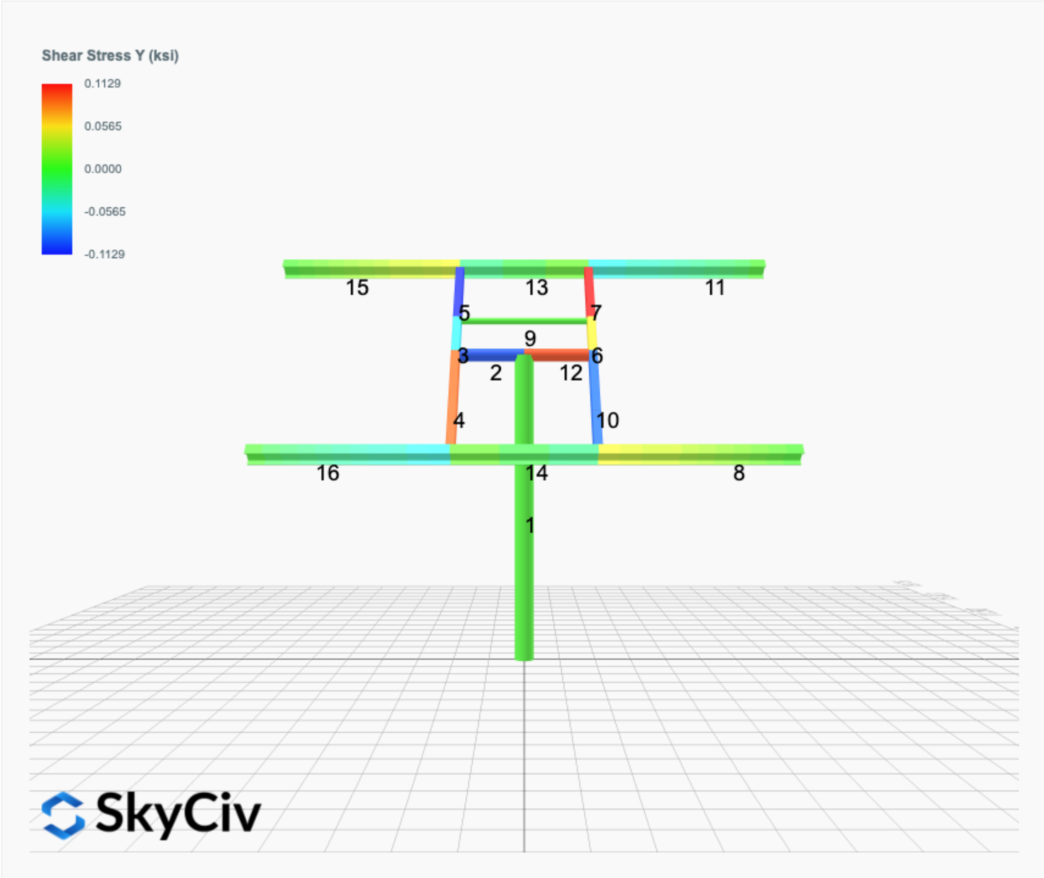
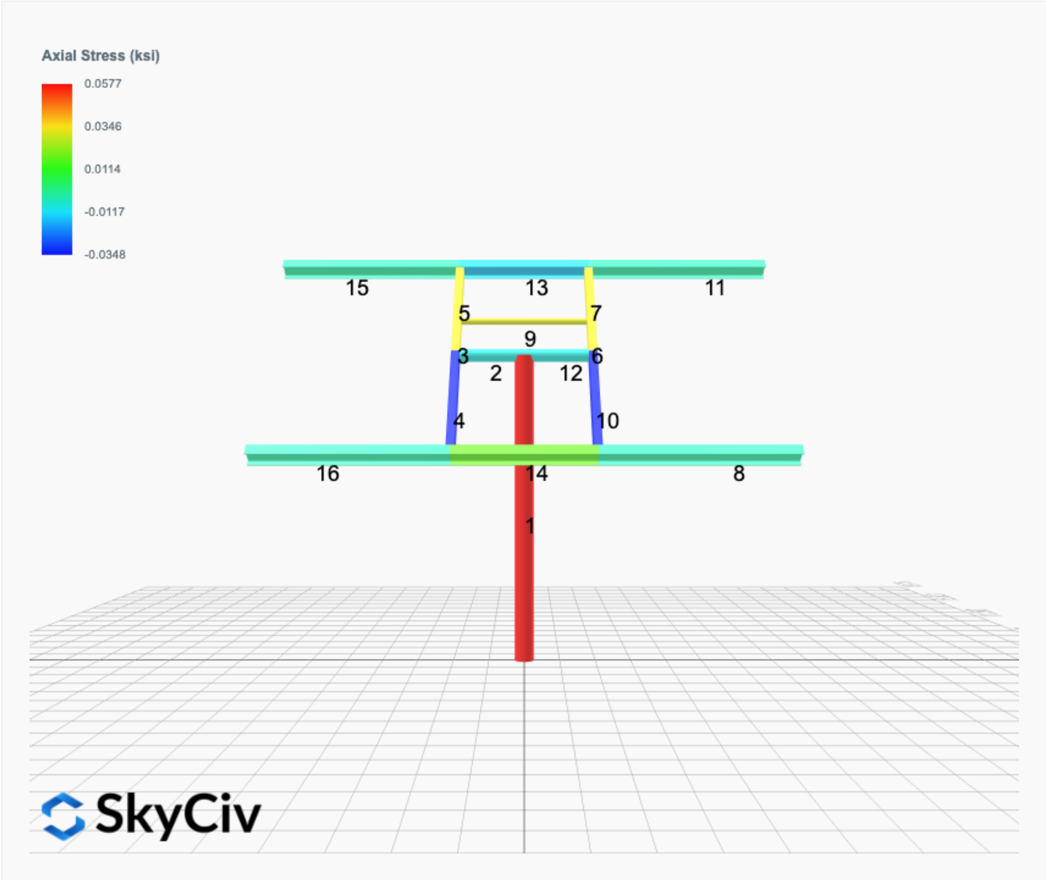
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
1.3365
0.6683
0.0000
-0.6683
-1.3365
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
0.1129
0.0565
0.0000
-0.0565
-0.1129
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
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0.0346
0.0114
-0.0117
-0.0348
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1

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	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 2. D + L	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 3. D + (S or Lr or R)	0.0000	1.7607	0.0000	0.0000	-0.0000	0.0150
ULS: 3. D + (S or Lr or R)	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6802	0.0000	0.0000	-0.0000	0.0150
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 5b. D + 0.7E	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.6802	0.0000	0.0000	-0.0000	0.0150
ULS: 8. 0.6D + 0.7E	0.0000	0.8632	0.0000	0.0000	-0.0000	0.0089
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4061	2.6185	0.0000	0.0000	-0.0000	12.6136
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4061	0.2587	0.0000	0.0000	-0.0000	-12.3563
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0546	2.5651	0.0000	0.0000	-0.0000	9.4641
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.6802	0.0000	0.0000	-0.0000	0.0150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0546	0.7952	0.0000	0.0000	-0.0000	-9.2634
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.6802	0.0000	0.0000	-0.0000	0.0150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0546	2.3235	0.0000	0.0000	-0.0000	9.4639
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0546	0.5537	0.0000	0.0000	-0.0000	-9.2635
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.4386	0.0000	0.0000	-0.0000	0.0148
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4061	2.0431	0.0000	0.0000	-0.0000	12.6077
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.8632	0.0000	0.0000	-0.0000	0.0089
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4061	-0.3167	0.0000	0.0000	-0.0000	-12.3622
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.8632	0.0000	0.0000	-0.0000	0.0089

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	3.8538
Shear X	-2.3435
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	21.2209

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	2.6185
Shear X	-1.4061
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	12.6136

Project Details

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

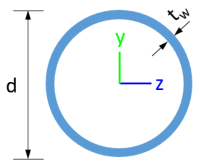
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

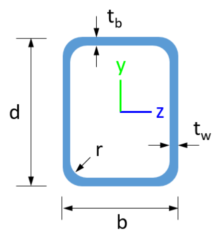
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

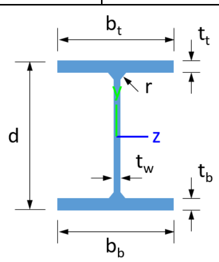
Section Dimensions



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)

1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	18.64	18.64	8.88	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.18,1.15,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.18,1.19	300	200	1
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.62,1.68,1.67,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.64,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.18,1.15,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.64,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
8	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.62,1.68,1.67,1.69,1.64,1.69,1.67,1.69,1.66,1.69	300	200	1
11	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
14	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
15	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	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	251.16	121.54	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.89	10.99	6.26	29.14	16.61
4	79.65	72.84	10.99	6.26	29.14	16.61
5	79.65	74.30	10.99	6.26	29.14	16.61
6	79.65	74.89	10.99	6.26	29.14	16.61
7	79.65	74.30	10.99	6.26	29.14	16.61
8	120.60	54.44	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.84	10.99	6.26	29.14	16.61
11	120.60	54.44	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.03	18.35	6.45	30.09	45.74
14	120.60	84.03	18.33	6.45	30.09	45.74

15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	23.36	6.45	30.09	45.74

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.032	0.502	0.000	0.031	0.000	0.518	#13	0.498	Not Required	Pass
2	0.001	0.187	0.147	0.043	0.027	0.334	#13	0.034	Not Required	Pass
3	0.005	0.350	0.039	0.035	0.006	0.384	#13	0.044	Not Required	Pass
4	0.005	0.348	0.079	0.035	0.014	0.396	#13	0.078	Not Required	Pass
5	0.005	0.216	0.082	0.035	0.016	0.232	#13	0.073	Not Required	Pass
6	0.005	0.350	0.039	0.035	0.006	0.384	#13	0.044	Not Required	Pass
7	0.005	0.216	0.082	0.035	0.016	0.232	#13	0.073	Not Required	Pass
8	0.000	0.040	0.052	0.017	0.004	0.082	#13	Not Required	Not Required	Pass
9	0.004	0.025	0.033	0.001	0.000	0.059	#13	0.198	Not Required	Pass
10	0.005	0.348	0.079	0.035	0.014	0.396	#13	0.078	Not Required	Pass
11	0.000	0.040	0.052	0.017	0.004	0.082	#13	Not Required	Not Required	Pass
12	0.001	0.187	0.147	0.043	0.027	0.334	#13	0.034	Not Required	Pass
13	0.002	0.116	0.112	0.025	0.006	0.201	#13	0.177	Not Required	Pass
14	0.003	0.119	0.112	0.025	0.006	0.201	#13	0.177	Not Required	Pass
15	0.000	0.040	0.052	0.017	0.004	0.082	#13	Not Required	Not Required	Pass
16	0.000	0.040	0.052	0.017	0.004	0.082	#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/comression)
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

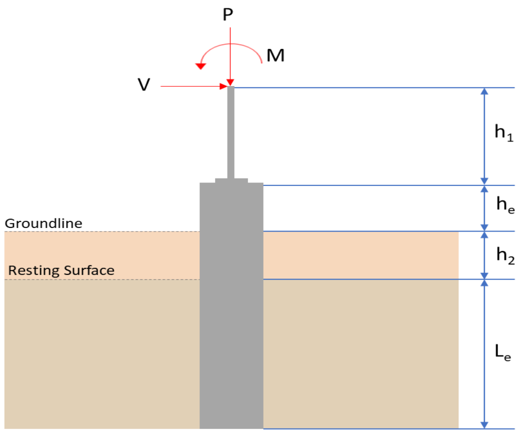
SkyCiv Foundation Design

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
$L = 5$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	2.618	3.854
V_x (kip)	-1.406	-2.344
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	12.614	21.221

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength,

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:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.406\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.22389\ \text{kip/ft}$$

	M_o - Moment per length of pile, $$M_o = \frac{M_z + (V_x\, H)}{1.57\, D}$$ $$M_o = \frac{(12.614 \text{ kipft}) + ((-1.406 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$ $$M_o = 2.0086 \text{ kipft/ft}$$ 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: $L_{e,x} = 4.6202 \text{ ft}$ - 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, $$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$ $$L_{e,req} = MAX[(4.6202 \text{ ft}), (0 \text{ ft})]$$ $$L_{e,req} = 4.62 \text{ ft}$$ L_e - Actual embedded length of pile, $$L_e = L - h_e - h_2$$ $$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$ $$L_e = 5 \text{ ft}$$ $Ratio$ - Embedded depth $$Ratio = \frac{L_{e,req}}{L_e}$$ $$Ratio = \frac{(4.62 \text{ ft})}{(5 \text{ ft})}$$ $$Ratio = 0.924$$	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 $$q = \frac{P_v}{A}$$ $$q = \frac{(2.618 \text{ kip})}{(16 \text{ ft}^2)}$$ $$q = 0.16362 \text{ kip/ft}^2$$ **Check bearing capacity ratio:** $Ratio$ - Capacity $$Ratio = \frac{q}{q_a}$$ $$Ratio = \frac{(0.16362 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$ $$Ratio = 0.081812$$	Status: **PASS** Ratio: **0.080**
Czerniak	**Lateral Soil Pressure (ASD):** L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.25$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.22389\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.0086\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0086\ \mathrm{kipft/ft}) \times (5\ \mathrm{ft})) + \left(3 \times (-0.22389\ \mathrm{kip/ft}) \times (5\ \mathrm{ft})^2\right)}{(6 \times (2.0086\ \mathrm{kipft/ft})) + (4 \times (-0.22389\ \mathrm{kip/ft}) \times (5\ \mathrm{ft}))}$$

$$a = 3.4462\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.0086\ \mathrm{kipft/ft})) + (3 \times (-0.22389\ \mathrm{kip/ft}) \times (5\ \mathrm{ft}))]^{\ 2}}{(5\ \mathrm{ft})^2 \times [(3 \times (2.0086\ \mathrm{kipft/ft})) + (2 \times (-0.22389\ \mathrm{kip/ft}) \times (5\ \mathrm{ft}))]}$$

$$p = 0.17322\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.0086\ \mathrm{kipft/ft})) + ((-0.22389\ \mathrm{kip/ft}) \times (5\ \mathrm{ft}))]}{(5\ \mathrm{ft})^2}$$

$$s = 0.69546\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.4462\ \mathrm{ft})}{2}$$

$$p_a = 0.25847\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.17322\ \mathrm{kip/ft^2}\right)}{\left(0.25847\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.67019$$

Status: **PASS**
Ratio: **0.670**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5\ \mathrm{ft})$$

$$p_s = 0.75\ \mathrm{kip/ft^2}$$

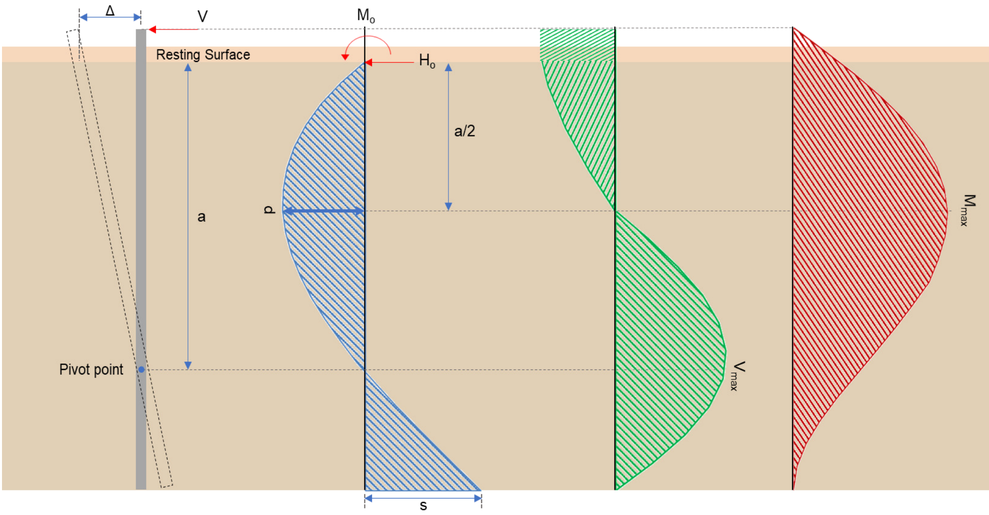
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.69546\ \mathrm{kip/ft^2}\right)}{\left(0.75\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.92729$$

Status: PASS
Ratio: 0.930



Pile Rotation Pressure Diagram Shear Diagram Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.344\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.37325\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.221\text{ kipft}) + ((-2.344\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 3.3791\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3791\text{ kipft/ft})}{(-0.37325\text{ kip/ft})}$$

$$E = 9.0533\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3791\text{ kipft/ft}) \times (5\text{ ft})) + \left(3 \times (-0.37325\text{ kip/ft}) \times (5\text{ ft})^2\right)}{(6 \times (3.3791\text{ kipft/ft})) + (4 \times (-0.37325\text{ kip/ft}) \times (5\text{ ft}))}$$

$$a = 3.4455\text{ ft}$$

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

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

$$V_{max} = ((-0.37325\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.0533\text{ ft})}{(5\text{ ft})} + 3\right) \times \left(\frac{(3.4455\text{ ft})}{(5\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.0533\text{ ft})}{(5\text{ ft})} + 2\right) \times \left(\frac{(3.4455\text{ ft})}{(5\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 5.7685\text{ kip}$$

$$V_{max} = 5.7085 \text{ kip}$$

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

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

$$M_{max} = ((-0.37325 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(9.0533 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4455 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.0533 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4455 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.0533 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4455 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.743 \text{ kipft}$$

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,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1 $A_{st,required}$

$$A_{st,required} = Min \left[\frac{\frac{P}{\phi \, \alpha} - (0.85 \, f'_{ck} \, A_g)}{f_{yk} - (0.85 \, f'_{ck})}, (0.08 \, A_g) \right]$$

$$A_{st,required} = Min \left[\frac{\frac{(3.854 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.468 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

$$A_{min} = Max \left[A_{st,required}, (0.0018 \, A_g) \right]$$

$$A_{min} = Max \left[(-84.468 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2)) \right]$$

$$A_{min} = 4.1472 \text{ in}^2$$

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

$$Ratio = \frac{A_{min}}{A_{st}}$$

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = Max \left[1.5, (1.5 \, d_{bar}) \right]$$

$$s_{rebar} = Max\left[1.5, (1.5 \times (0.625\text{ in}))\right]$$

$$s_{rebar} = 1.5\text{ in}$$

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

$$s_{ties} = Min\left[(16\,d_{bar}), (48\,d_{ties}), Min\,(D, b)\right]$$

$$s_{ties} = Min\left[(16 \times (0.625\text{ in})), (48 \times (0.375\text{ in})), Min\,((48\text{ in}), (48\text{ in}))\right]$$

$$s_{ties} = 10\text{ in}$$

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi\, 0.80\ \left[\left(0.85\, f'_{ck}\ [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$

$$\phi P_N = 2675.2\text{ kip}$$

$Ratio$ - Capacity

$$Ratio = \frac{P}{\phi P_N}$$

$$Ratio = \frac{(3.854\text{ kip})}{(2675.2\text{ kip})}$$

$$Ratio = 0.0014406$$

Status: **PASS**
Ratio: **0.000**

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\text{ in}$ - Effective width,

22.5.2.2 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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,

22.5.5.1.1 $V_{c,max}$ - 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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 3.854\text{ kip} \rightarrow 3854\text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \frac{P}{6\,A_g}\right]\ b_w\,d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(3854 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (119 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.1.2 $V_{s,a}$ - 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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\, \left(V_c + V_s\right)$$

$$\phi V_n = (0.65) \times ((119 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.43 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.7685 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(5.7685 \text{ kip})}{(110.43 \text{ kip})}$$

$$Ratio = 0.052236$$

Status: **PASS**
Ratio: **0.050**

	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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\,0.85\,f_{ck}'\,S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\text{ ksi}) \times \left(18432\text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6\text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\text{ kipft}), (2121.6\text{ kipft})]$$ $$\phi M_n = 249.6\text{ kipft}$$ **Considering x-direction:** $M_{max} = 13.743\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(13.743\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.05506$$	Status: **PASS** Ratio: **0.060**