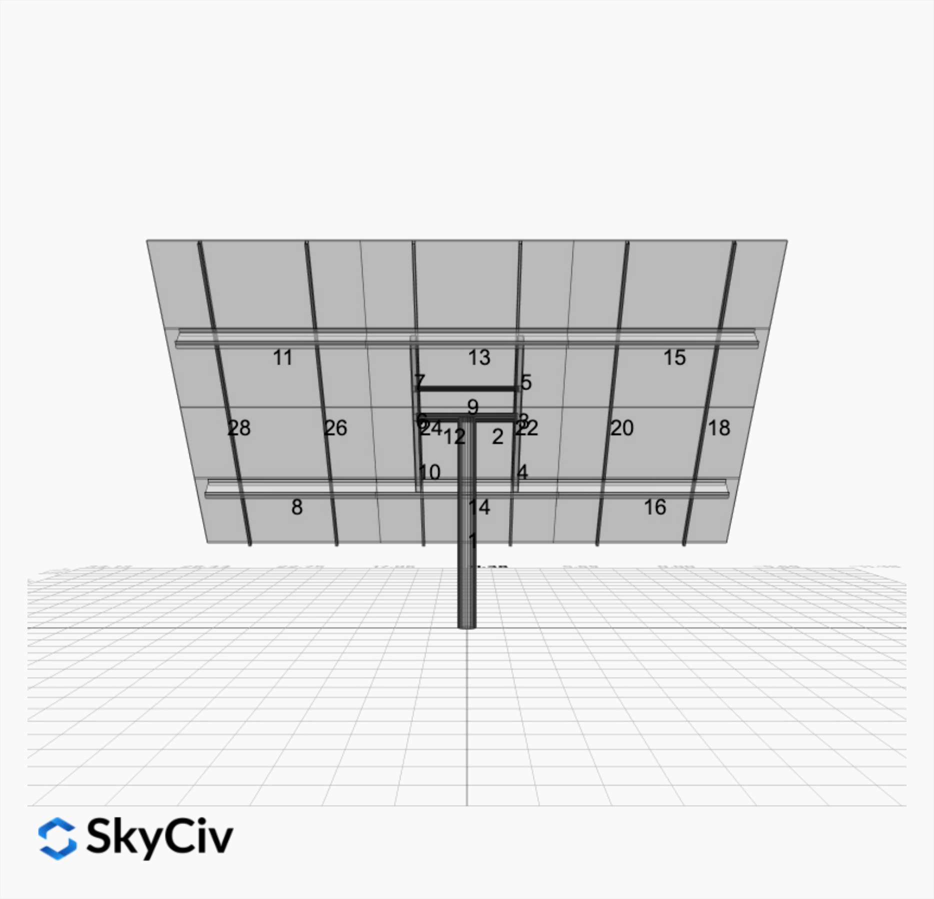
Project Details

Project Name: MTSOLAR_6GG7FDEHEF218 - V1Jb **Date:** Fri Oct 25 2024

Location: Soldotna, AK 99669, USA **Number of Modules:** 12

Unique ID: 1P-0-8TOP-XD-84-L-4Hx3W-5G37 **Number of Poles:** 1

Dealer: ______________________________ **Date Sold:** ______________________________



Array Dimensions N/S	15.00 ft
Array Dimensions E/W	22.75 ft
Winter Tilt Angle	50
Front Edge Clearance	2.5 ft

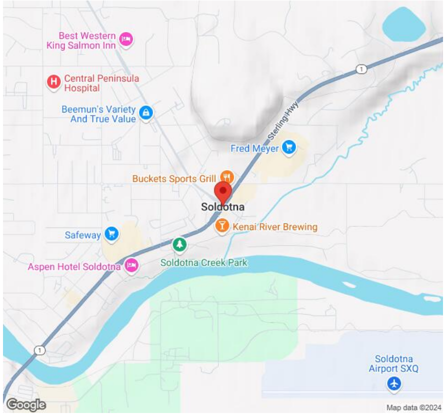
MT Solar Bill of Materials (1P-0-8TOP-XD-84-L-4Hx3W-5G37)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-XD	H-Frame Assembly-XD	1
MTS-XD-Wing-84	84IN XD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (178in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12
Ground Lug	3

Site Details:



Site Address: Soldotna, AK 99669, USA

Array Specification

Duty Classification:	XD
Module Width:	44.50 in
Module Length:	90.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	50
Front Edge Clearance:	2.5
Total Array Height at Tilt:	13.99 ft
Total Frame Length:	21.50 ft
Frame Weight:	1278 lbs
Array Dimensions N/S:	15.00 ft
Array Dimensions E/W:	22.75 ft
Rail Length:	180.00 in
Rail Spacing:	3.79 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.25 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: 5.25 ft
Foundation Volume:	3.111 y^3

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Soldotna, AK 99669, USA
Wind Speed:	100 mph
Snow Load:	50 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":100,"snow_load_override":50,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"","designer_email":"mark@ak.solar","designer_phone":"","project_id":"MTSOLAR_6GG7FDEHEF218 - V1Jb","site_address":"Soldotna, AK 99669, USA","module_width":44.5,"module_length":90,"number_rows":"4","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":2.5,"risk_category":"I","exposure_category":"B","frame_duty_override":"XD","pole_override":"8_40","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}
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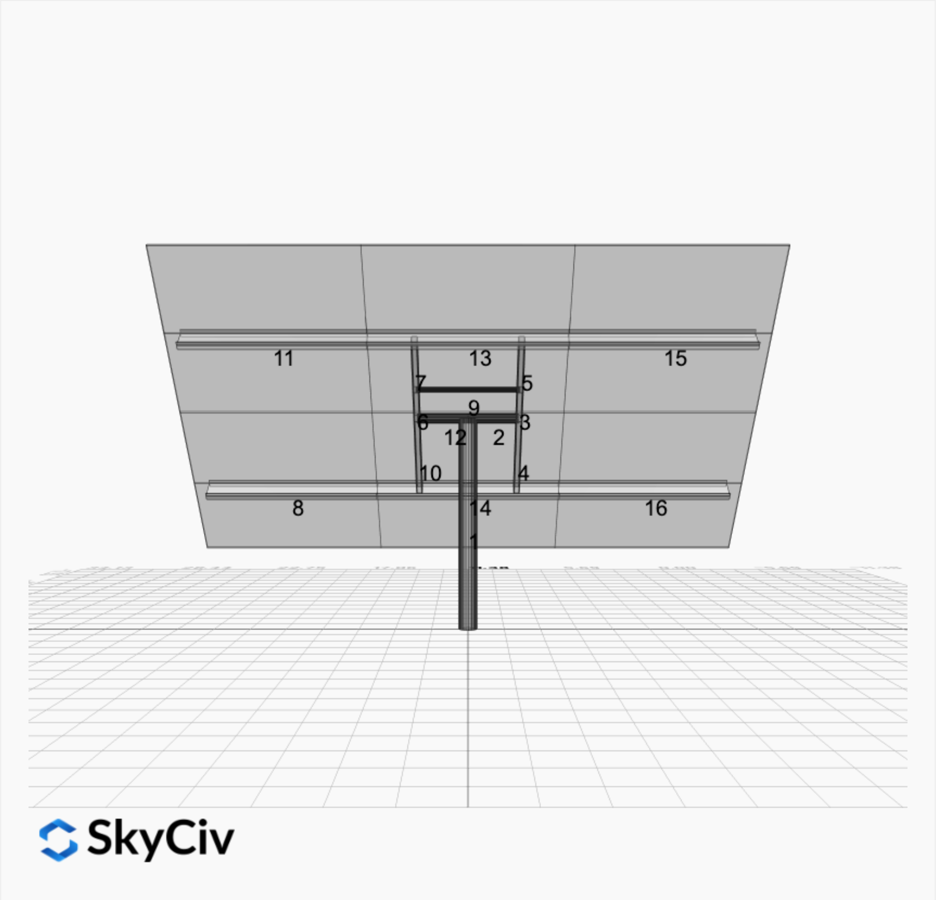
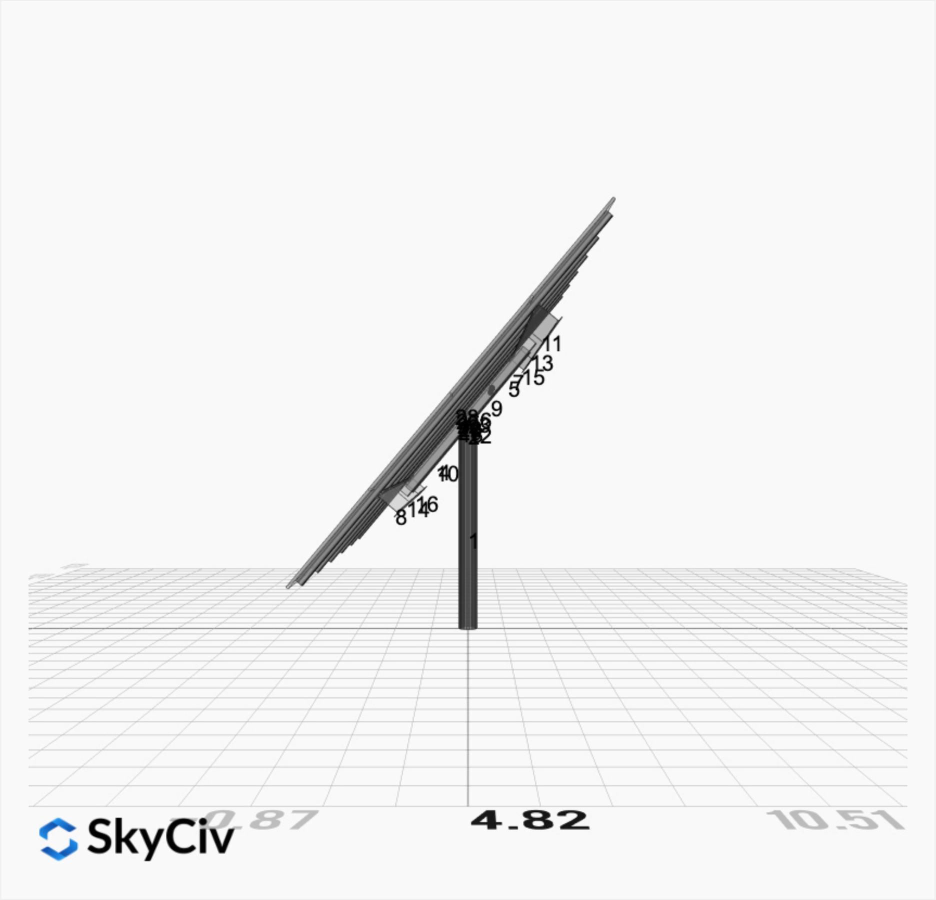
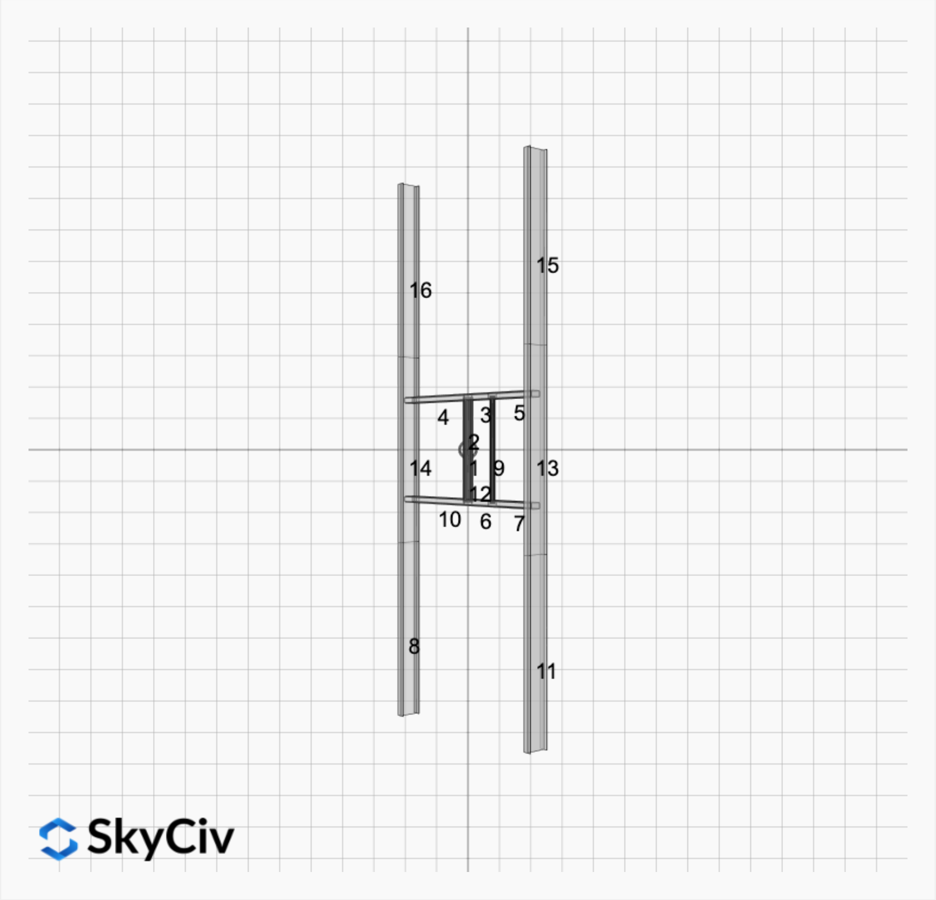
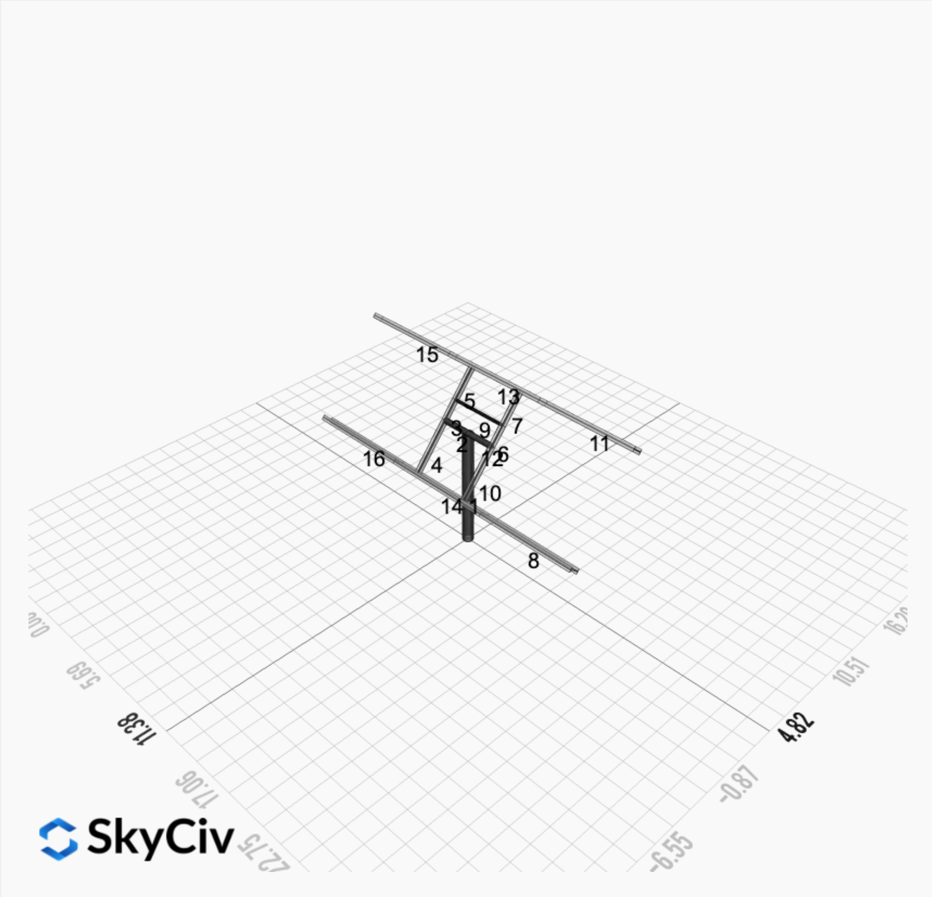
Design Notes:

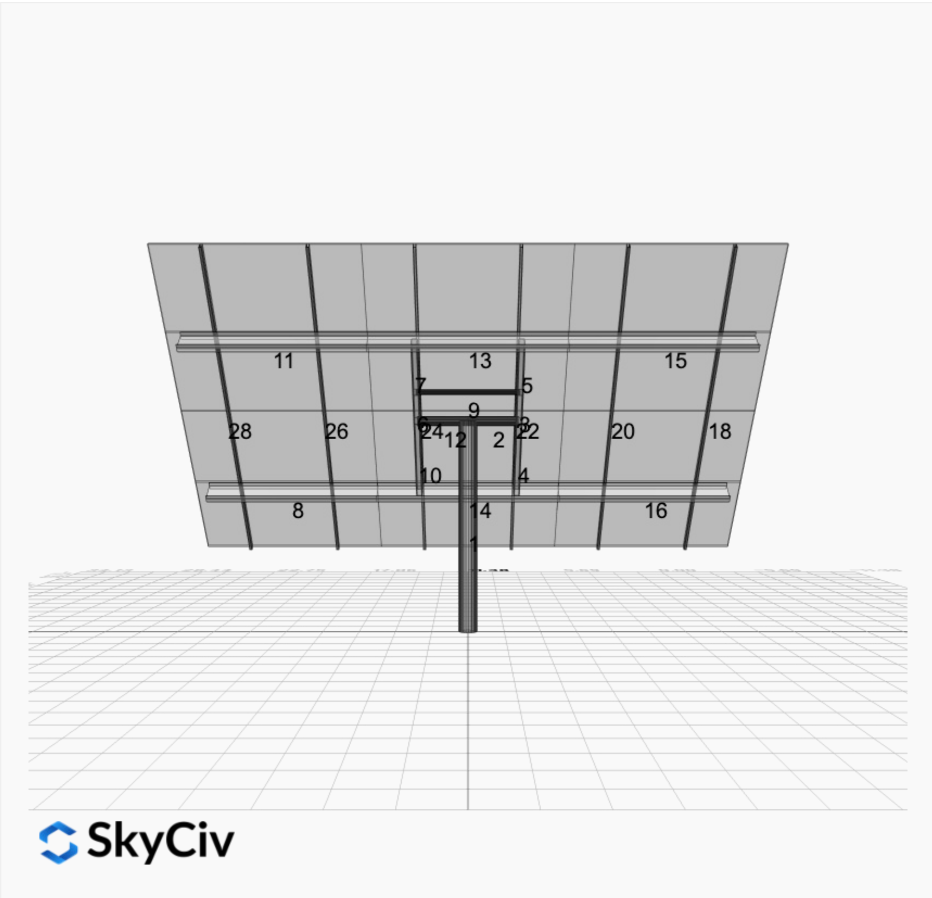
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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




0.87
4.82
10.51




15
13
5
7
9
3
2
11
16
4
12
6
10
14
1
8
5.69
11.38
17.06
22.75
10.51
4.82
-0.87
-6.55
SkyCiv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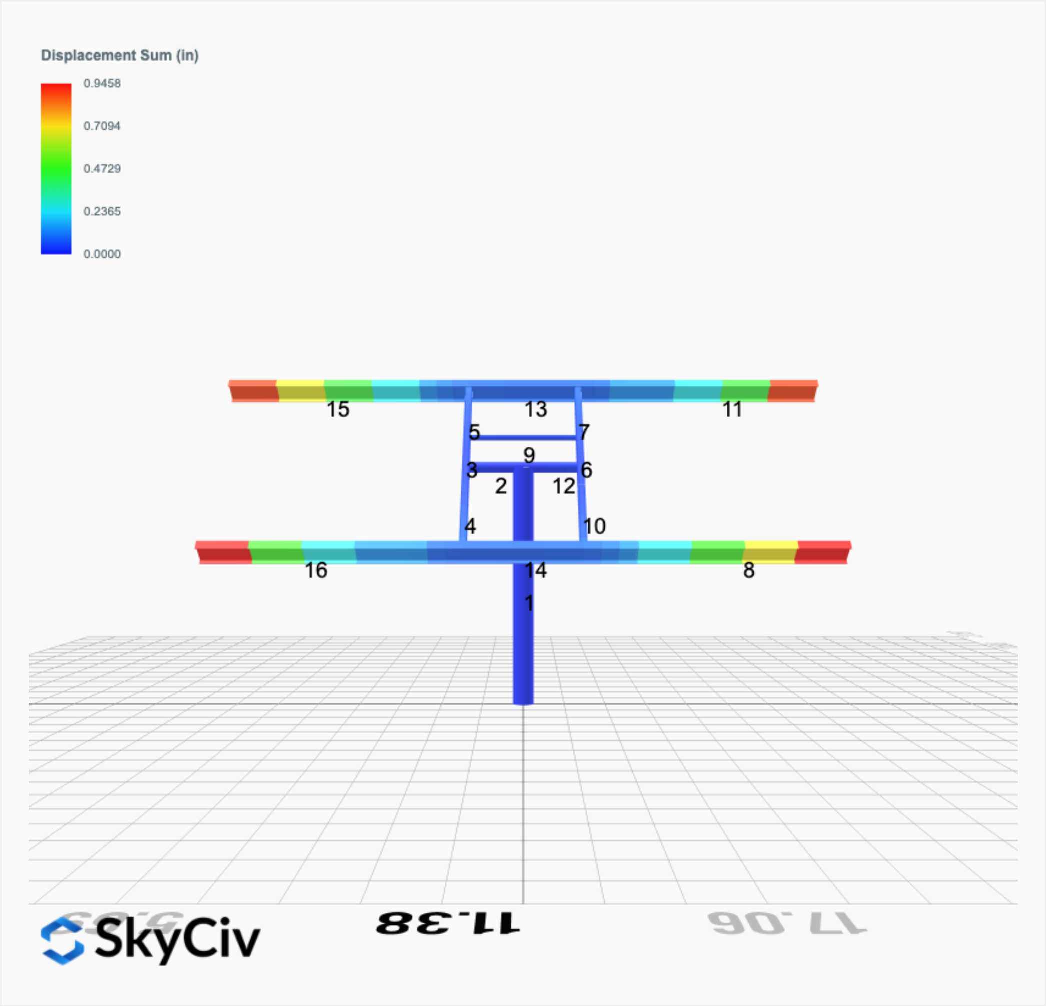
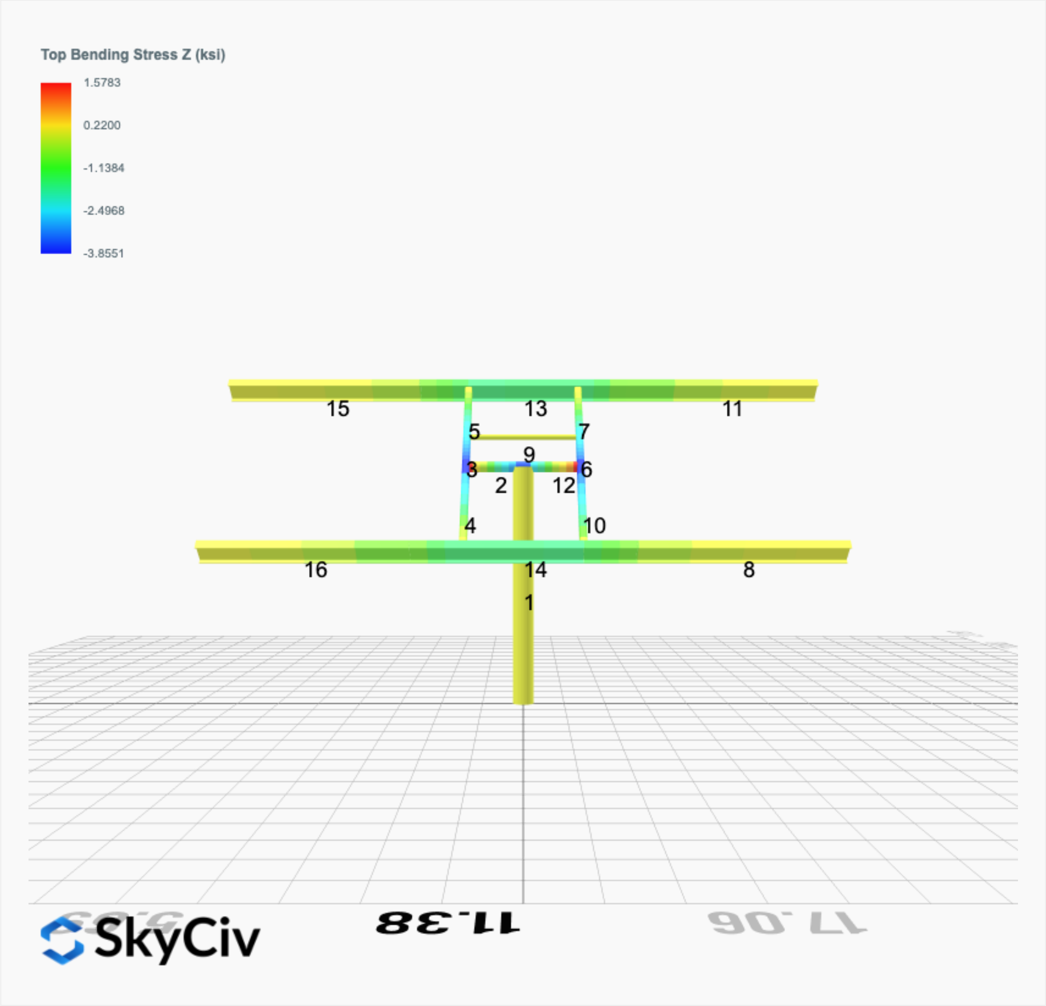
13
15
7
5
9
6
3
28
26
24
12
2
22
20
18
10
4
8
14
16
1
SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
1.5783
0.2200
-1.1384
-2.4968
-3.8551
15
13
11
5
7
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3
6
2
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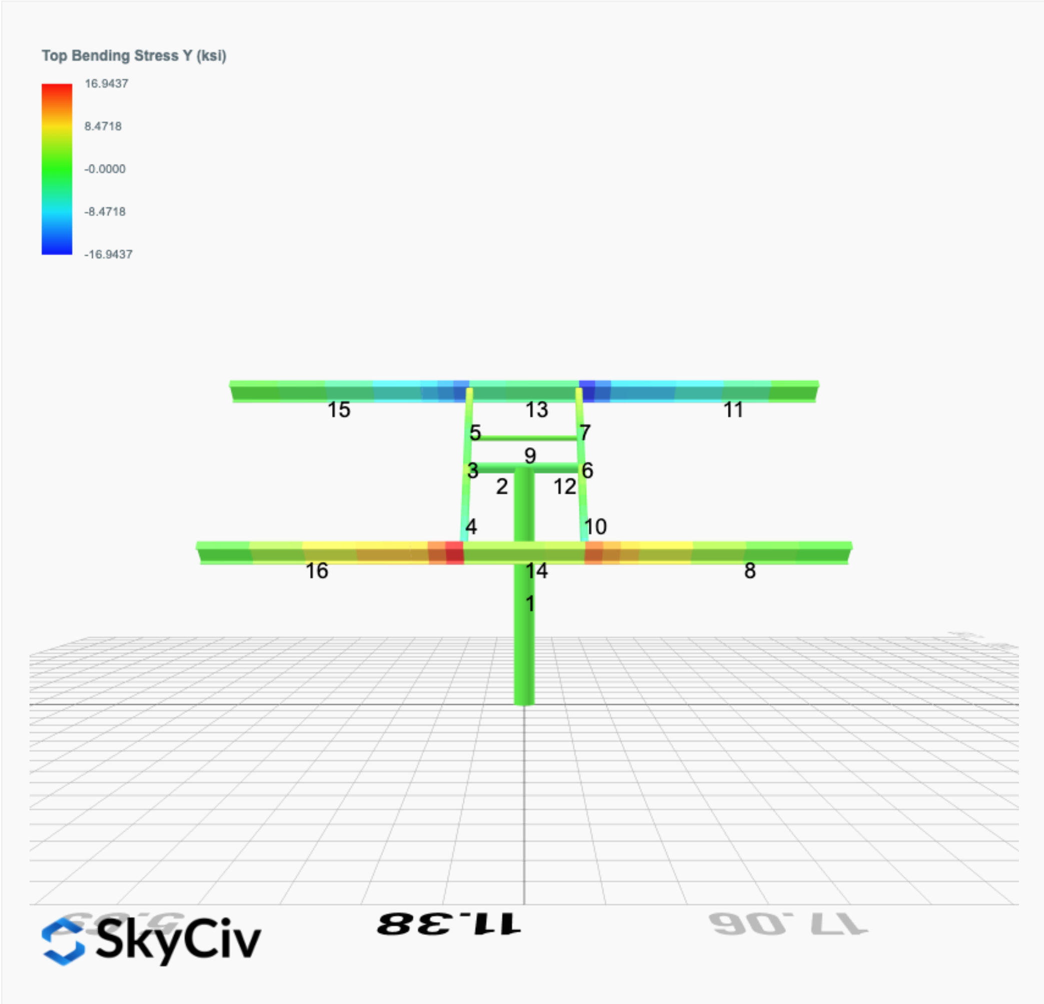
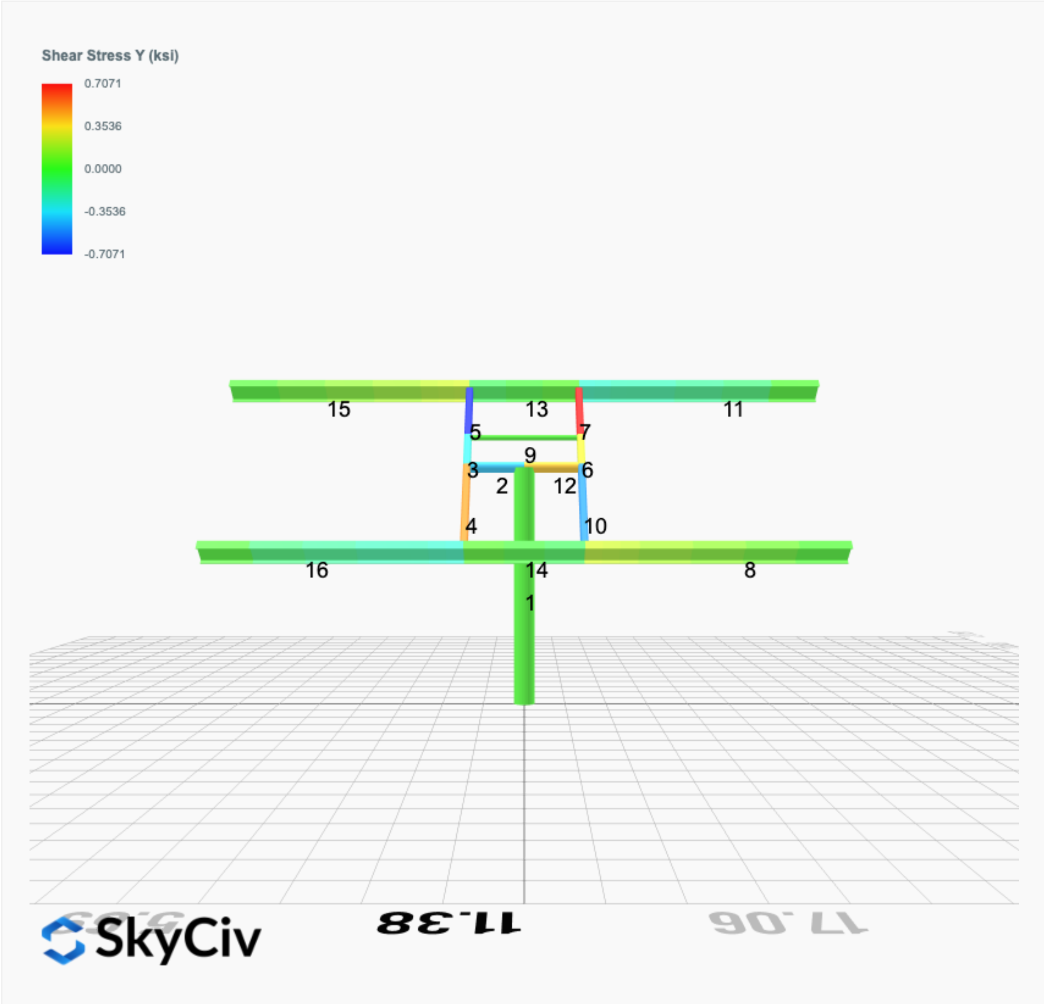
4
10
16
14
8
1


Top Bending Stress Y (ksi)
16.9437
8.4718
-0.0000
-8.4718
-16.9437
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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0.3536
0.0000
-0.3536
-0.7071
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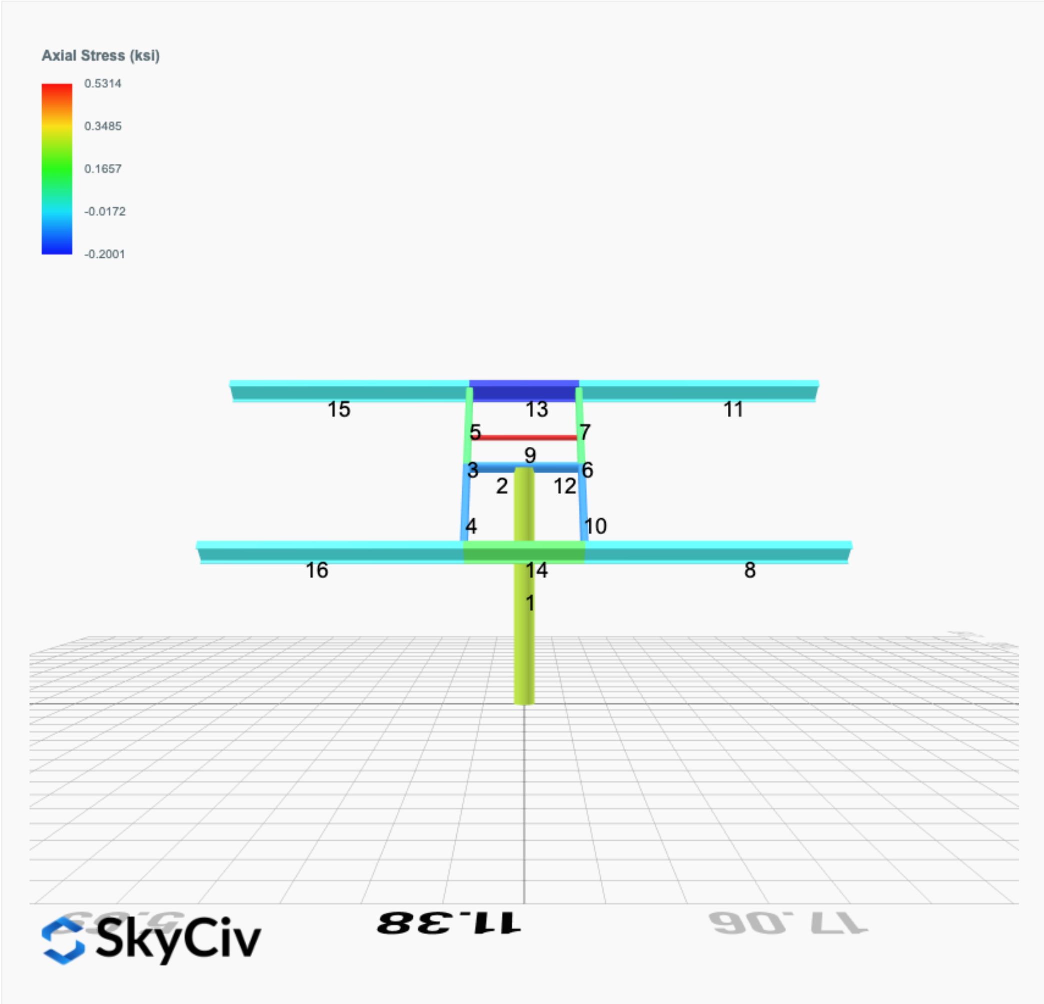
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.5314
0.3485
0.1657
-0.0172
-0.2001
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
11.38
17.06

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	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 2. D + L	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 3. D + (S or Lr or R)	-0.0000	4.8488	0.0000	0.0000	-0.0000	0.0287
ULS: 3. D + (S or Lr or R)	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.2789	0.0000	0.0000	-0.0000	0.0274
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 5b. D + 0.7E	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.2789	0.0000	0.0000	-0.0000	0.0274
ULS: 8. 0.6D + 0.7E	-0.0000	1.5416	0.0000	0.0000	-0.0000	0.0141
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.9447	4.2011	0.0000	0.0000	-0.0000	16.1263
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.9447	0.9375	0.0000	0.0000	-0.0000	-15.9439
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4585	5.5028	0.0000	0.0000	-0.0000	12.1045
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	4.2789	0.0000	0.0000	-0.0000	0.0274
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4585	3.0551	0.0000	0.0000	-0.0000	-11.9481
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	4.2789	0.0000	0.0000	-0.0000	0.0274
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.4585	3.7931	0.0000	0.0000	-0.0000	12.1006
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4585	1.3454	0.0000	0.0000	-0.0000	-11.9520
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.5693	0.0000	0.0000	-0.0000	0.0235
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.9447	3.1734	0.0000	0.0000	-0.0000	16.1169
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.5416	0.0000	0.0000	-0.0000	0.0141
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.9447	-0.0902	0.0000	0.0000	-0.0000	-15.9533
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.5416	0.0000	0.0000	-0.0000	0.0141

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	8.0902
Shear X	-3.2412
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	27.0809

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	5.5028
Shear X	-1.9447
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	16.1263

Project Details

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

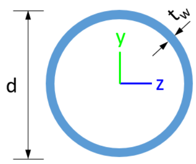
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

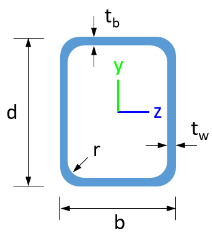
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

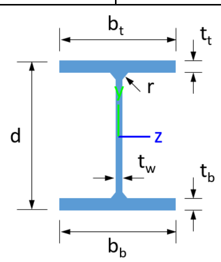
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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)

3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	17.32	17.32	8.25	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.14,1.19,1.18,1.19,1.17,1.19	300	200	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.65,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.68,1.68,1.68,1.69,1.60,1.69,1.67,1.69,1.66,1.69	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.63,1.68,1.67,1.68,1.66,1.68	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.14,1.19,1.18,1.19,1.17,1.19	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.63,1.68,1.67,1.68,1.66,1.68	300	200	1
8	20	7.00	7.00	7.00	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.65,1.68,1.67,1.69,1.65,1.69,1.67,1.68,1.68,1.68,1.68,1.69,1.60,1.69,1.67,1.69,1.66,1.69	300	200	1
11	20	7.00	7.00	7.00	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
15	20	7.00	7.00	7.00	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32	300	200	1
16	20	7.00	7.00	7.00	2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32,2.32	300	200	1

Member Design Capacity

Member ID	**Φ_tP_n (kip)**	**Φ_cP_n (kip)**	**Φ_bM_{zn} (k-ft)**	**Φ_bM_{yn} (k-ft)**	**Φ_vV_{yn} (kip)**	**Φ_vV_{zn} (kip)**
1	377.97	262.18	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	68.92	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	68.92	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.07	6.46	56.26	44.91
14	159.30	97.43	31.06	6.46	56.26	44.91

15	159.30	68.92	46.90	6.46	56.26	44.91
16	159.30	68.92	46.90	6.46	56.26	44.91

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	0.031	0.325	0.000	0.029	0.000	0.338	#13	0.354	Not Required	Pass
2	0.007	0.211	0.131	0.052	0.022	0.324	#13	0.036	Not Required	Pass
3	0.008	0.310	0.063	0.031	0.015	0.325	#13	0.046	Not Required	Pass
4	0.008	0.308	0.237	0.031	0.048	0.420	#21	0.082	Not Required	Pass
5	0.008	0.192	0.246	0.031	0.066	0.250	#6	0.076	Not Required	Pass
6	0.008	0.310	0.063	0.031	0.015	0.325	#13	0.046	Not Required	Pass
7	0.008	0.192	0.246	0.031	0.066	0.250	#6	0.076	Not Required	Pass
8	0.000	0.080	0.414	0.019	0.017	0.487	#21	Not Required	Not Required	Pass
9	0.035	0.020	0.036	0.001	0.000	0.067	#21	0.206	Not Required	Pass
10	0.008	0.308	0.237	0.031	0.048	0.420	#21	0.082	Not Required	Pass
11	0.000	0.080	0.414	0.019	0.017	0.487	#21	Not Required	Not Required	Pass
12	0.007	0.211	0.131	0.052	0.022	0.324	#13	0.036	Not Required	Pass
13	0.012	0.195	0.647	0.024	0.021	0.820	#21	0.204	Not Required	Pass
14	0.014	0.197	0.647	0.024	0.021	0.820	#21	0.204	Not Required	Pass
15	0.000	0.080	0.414	0.019	0.017	0.487	#21	Not Required	Not Required	Pass
16	0.000	0.080	0.414	0.019	0.017	0.487	#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/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

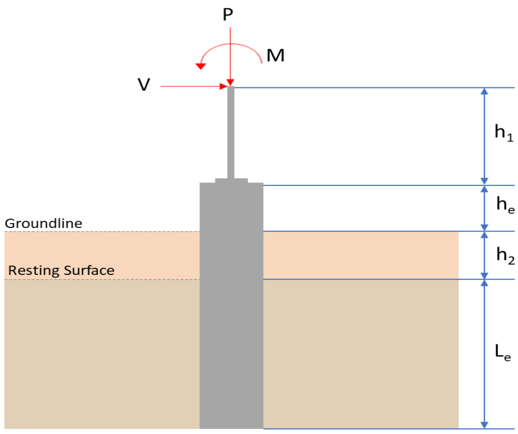
SkyCiv Foundation Design

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
$L = 5.25$ 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)	5.503	8.090
V_x (kip)	-1.945	-3.241
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	16.126	27.081

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,

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

$$H_o = \frac{(-1.945\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.30971\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.126\text{ kipft}) + ((-1.945\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 2.5678\text{ kipft/ft}$$

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:
$L_{e,x} = 4.8636\text{ ft}$ - 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,

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = MAX[(4.8636\text{ ft}), (0\text{ ft})]$$

$$L_{e,req} = 4.864\text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25\text{ ft}) - (0\text{ ft}) - (0\text{ ft})$$

$$L_e = 5.25\text{ ft}$$

$Ratio$ - Embedded depth

$$Ratio = \frac{L_{e,req}}{L_e}$$

$$Ratio = \frac{(4.864\text{ ft})}{(5.25\text{ ft})}$$

$$Ratio = 0.92648$$

Status: **PASS**
Ratio: **0.930**

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

$$q = \frac{P_v}{A}$$

$$q = \frac{(5.503\text{ kip})}{(16\text{ ft}^2)}$$

$$q = 0.34394\text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

$$Ratio = \frac{q}{q_a}$$

$$Ratio = \frac{\left(0.34394\text{ kip/ft}^2\right)}{(2000\text{ psf})}$$

$$Ratio = 0.17197$$

Status: **PASS**
Ratio: **0.170**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.3125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.30971\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.5678\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5678\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.30971\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (2.5678\ \mathrm{kipft/ft})) + (4 \times (-0.30971\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.6299\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.5678\ \mathrm{kipft/ft})) + (3 \times (-0.30971\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{\ 2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (2.5678\ \mathrm{kipft/ft})) + (2 \times (-0.30971\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = 0.17781\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.5678\ \mathrm{kipft/ft})) + ((-0.30971\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.76401\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.6299\ \mathrm{ft})}{2}$$

$$p_a = 0.27224\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.17781\ \mathrm{kip/ft^2}\right)}{\left(0.27224\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65313$$

Status: **PASS**
Ratio: **0.650**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.25\ \mathrm{ft})$$

$$p_s = 0.7875\ \mathrm{kip/ft^2}$$

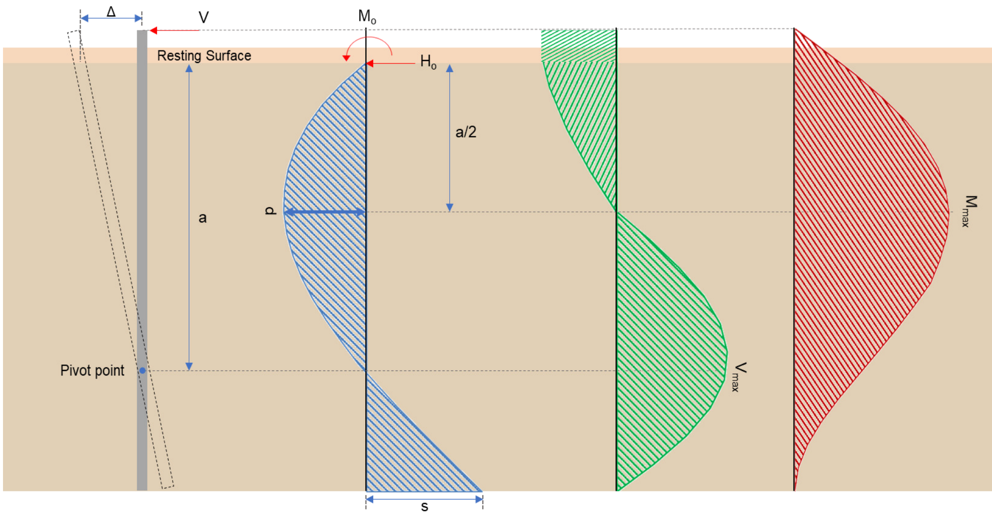
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.76401\ \mathrm{kip/ft^2}\right)}{\left(0.7875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.97017$$

Status: PASS
Ratio: 0.970



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.241\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.51608\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.081\ \text{kipft}) + ((-3.241\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.3123\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3123\ \text{kipft/ft})}{(-0.51608\ \text{kip/ft})}$$

$$E = 8.3558\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3123\ \text{kipft/ft}) \times (5.25\ \text{ft})) + \left(3 \times (-0.51608\ \text{kip/ft}) \times (5.25\ \text{ft})^2\right)}{(6 \times (4.3123\ \text{kipft/ft})) + (4 \times (-0.51608\ \text{kip/ft}) \times (5.25\ \text{ft}))}$$

$$a = 3.6292\ \text{ft}$$

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

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

$$V_{max} = ((-0.51608\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.3558\ \text{ft})}{(5.25\ \text{ft})} + 3\right) \times \left(\frac{(3.6292\ \text{ft})}{(5.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (8.3558\ \text{ft})}{(5.25\ \text{ft})} + 2\right) \times \left(\frac{(3.6292\ \text{ft})}{(5.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 7.175\ \text{kip}$$

$$V_{max} = 7.175 \text{ kip}$$

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

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

$$M_{max} = ((-0.51608 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(8.3558 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6292 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.3558 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6292 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.3558 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6292 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 17.851 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(8.09 \text{ kip})}{(0.65) \times (0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right) \right]$$

$$A_{st,required} = -84.327 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.327 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

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

$$Ratio = \frac{(8.09 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0030241$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.09 \text{ kip} \rightarrow 8090 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8090 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.56 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.56 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.56 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.56 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.175 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(7.175 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.064758$$

Status: **PASS**
Ratio: **0.060**

	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:** $M_{max} = 17.851\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(17.851\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.07152$$	Status: **PASS** Ratio: **0.070**