Project Details

Project Name: YNP - Lamar Buffalo Ranch 410w - v2cu

Location: VQW9+54C, Fossil Forest, WY 82190, USA

Unique ID: 1P-0-6TOP-SD-57-L-4Hx3W-LFI5

Dealer: ______________________________

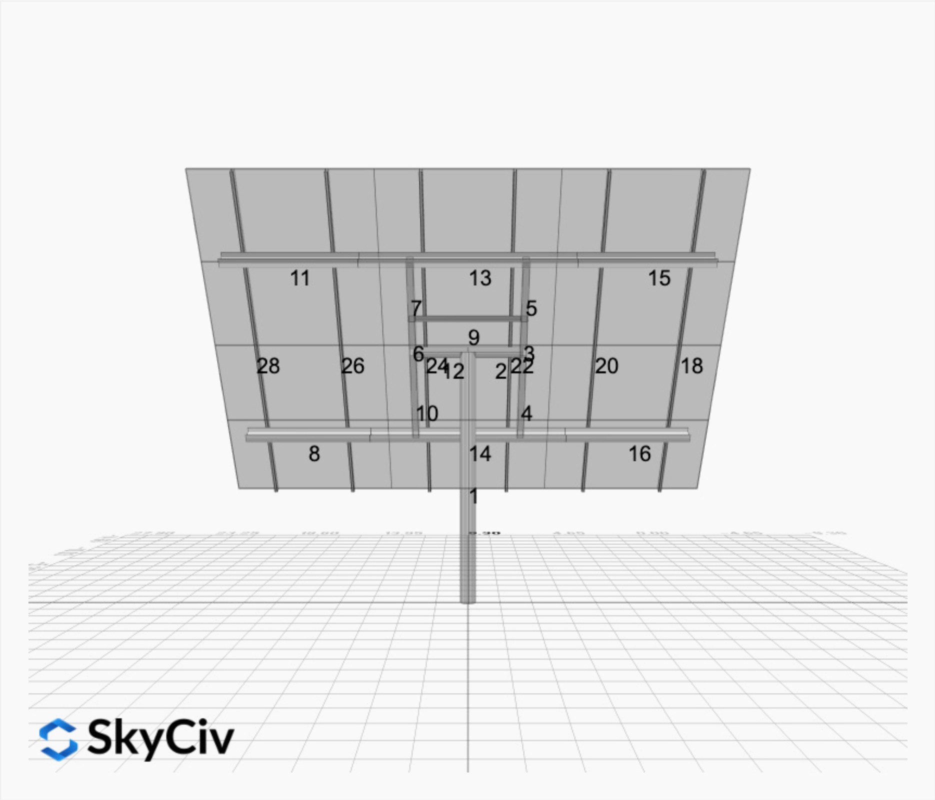
Date: Tue Mar 25 2025

Number of Modules: 12

Number of Poles: 1

Date Sold:

____



Array Dimensions N/S	13.67 ft
Array Dimensions E/W	18.60 ft
Winter Tilt Angle	55
Front Edge Clearance	3.5 ft

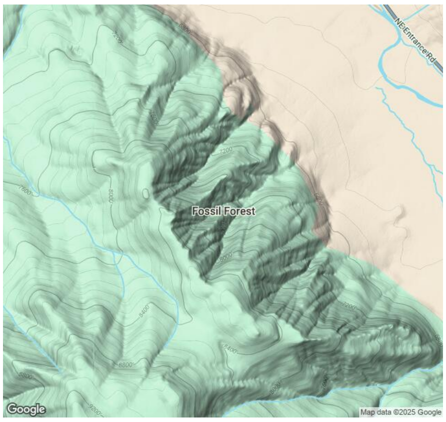
MT Solar Bill of Materials (1P-0-6TOP-SD-57-L-4Hx3W-LFI5)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-57	57IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (164in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12
Ground Lug	3

Site Details:



Site Address: VQW9+54C, Fossil Forest, WY 82190, USA

Array Specification

Duty Classification:	SD
Module Width:	40.50 in
Module Length:	73.40in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	55
Front Edge Clearance:	3.5
Total Array Height at Tilt:	14.70 ft
Total Frame Length:	17.00 ft
Module Info/Notes:	
Array Dimensions N/S:	13.67 ft
Array Dimensions E/W:	18.60 ft
Rail Length:	164.00 in
Rail Spacing:	3.10 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.10 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	60 x 60 in
Foundation Depth (below grade):	Pile 1: 5.25 ft
Foundation Volume:	4.861 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	VQW9+54C, Fossil Forest, WY 82190, USA
Wind Speed:	101 mph
Snow Load:	140 psf

Design Disclaimer

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

AutoDesigner Input

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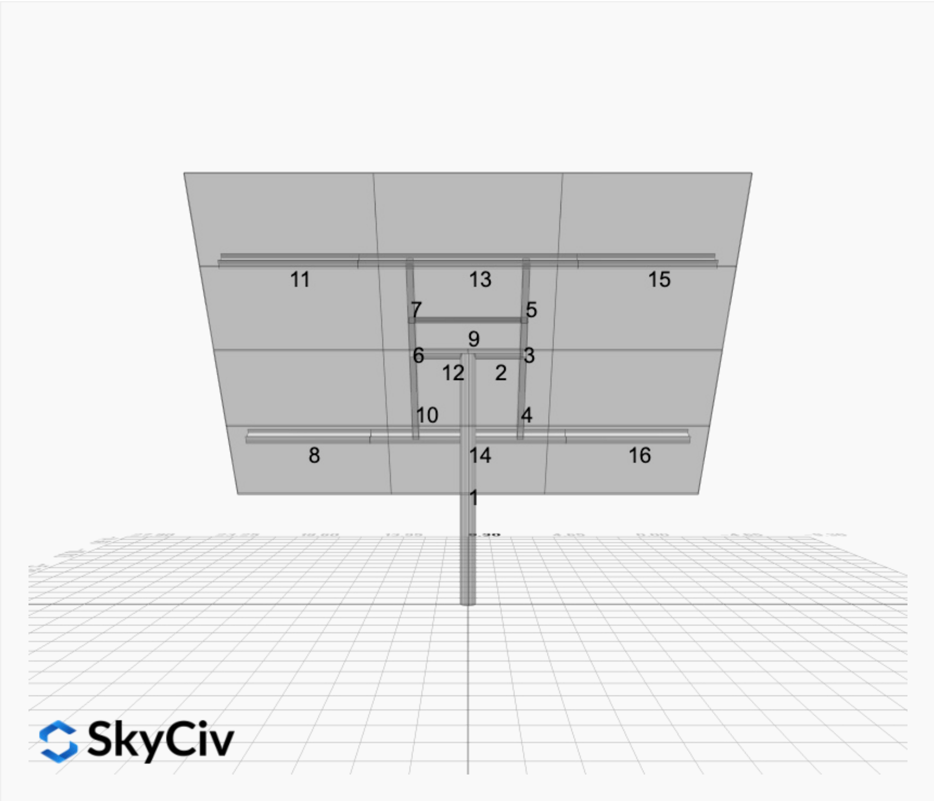
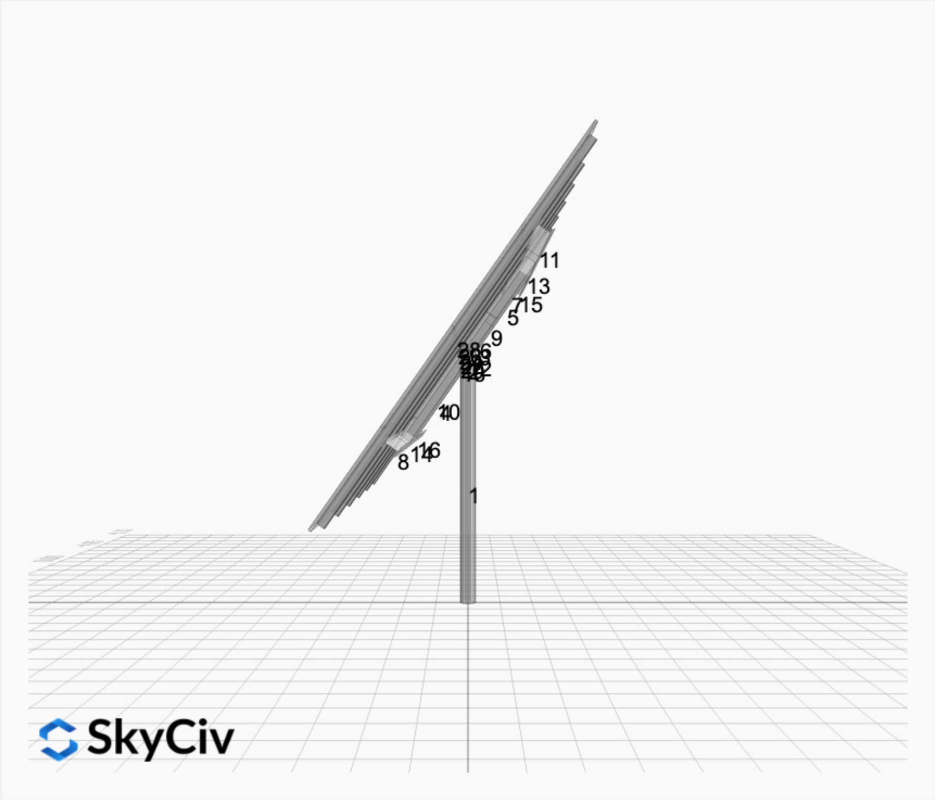
Design Notes:

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





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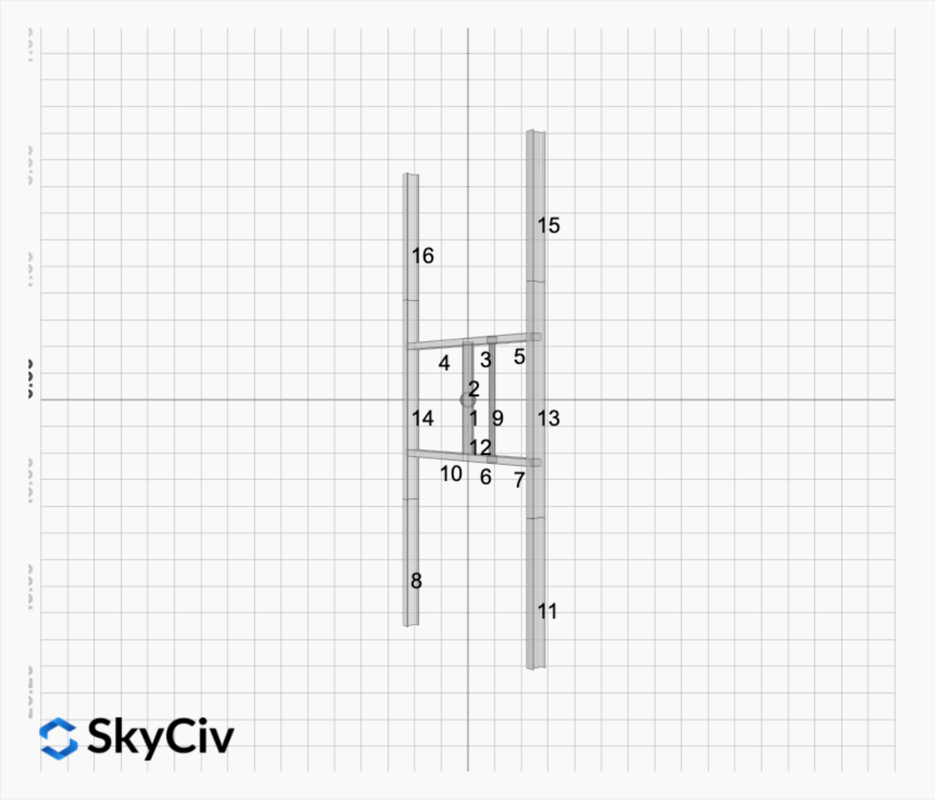
SkyCiv



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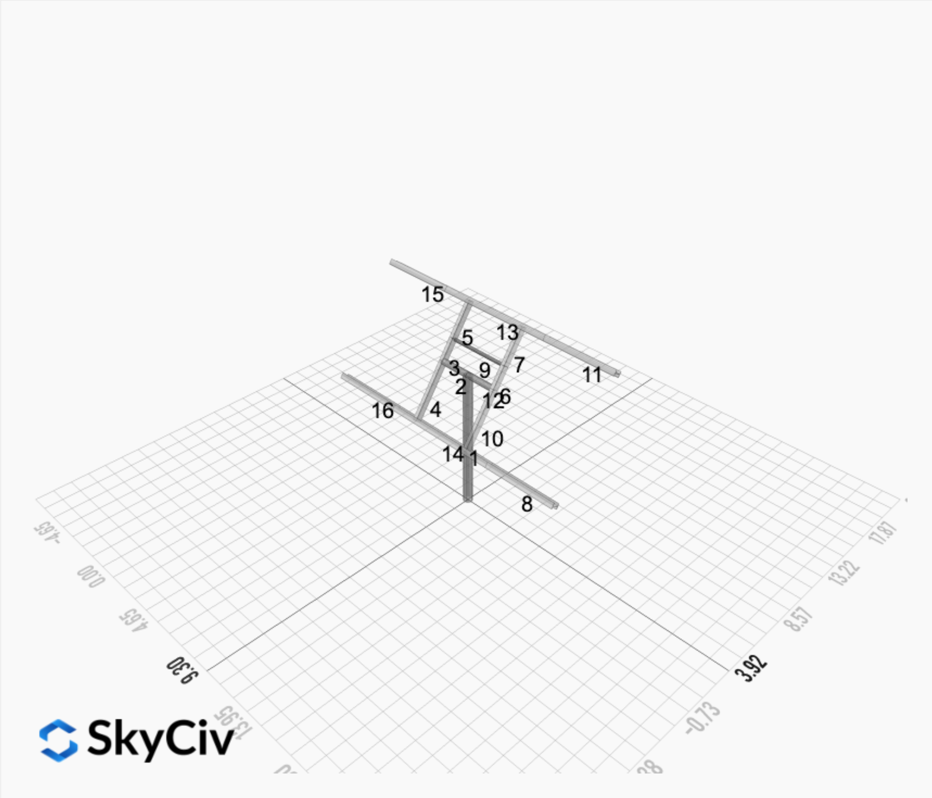
SkyCiv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4.65
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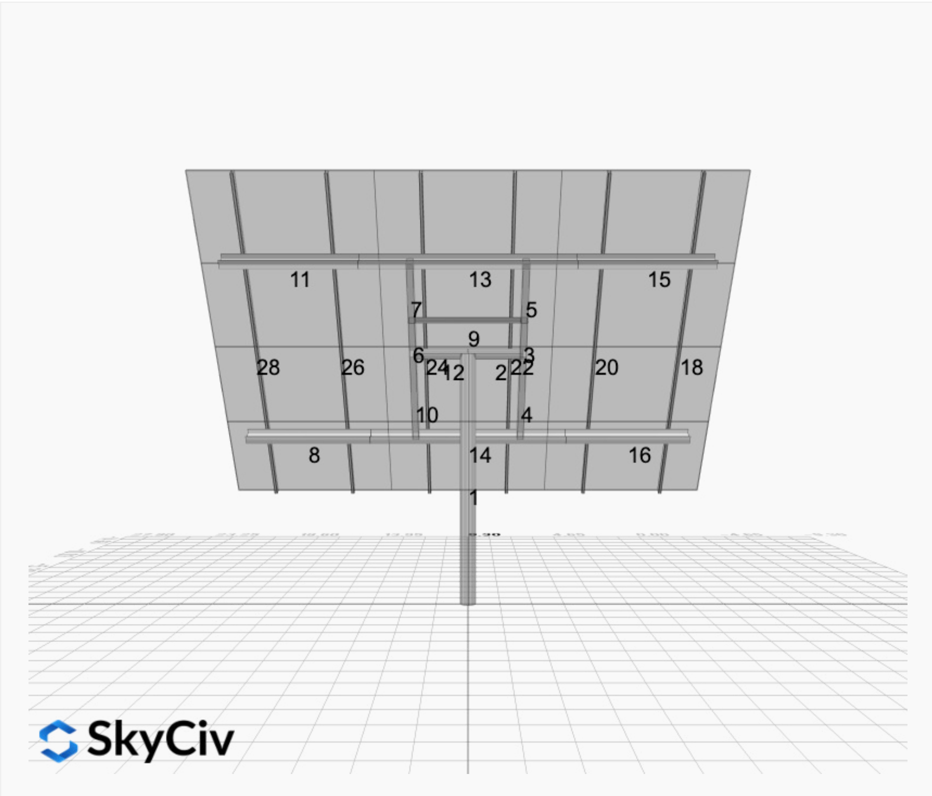
13.95
17.87
13.22
8.57
3.92
-0.73
SkyCiv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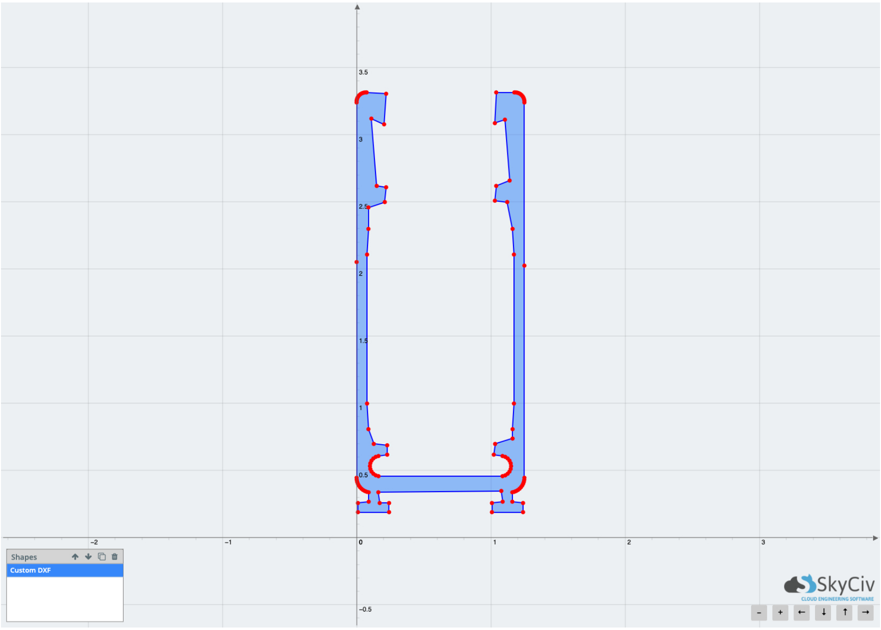
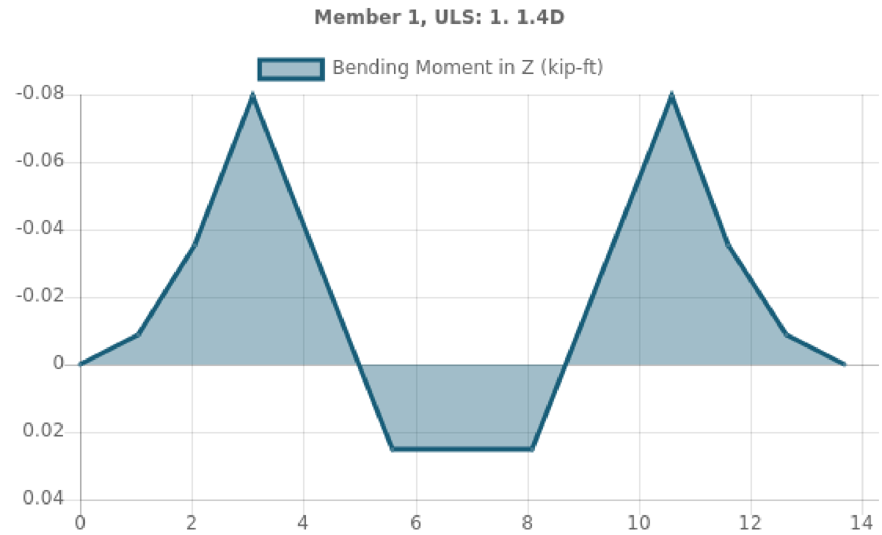
1
SkyCiv

Rail Design Check

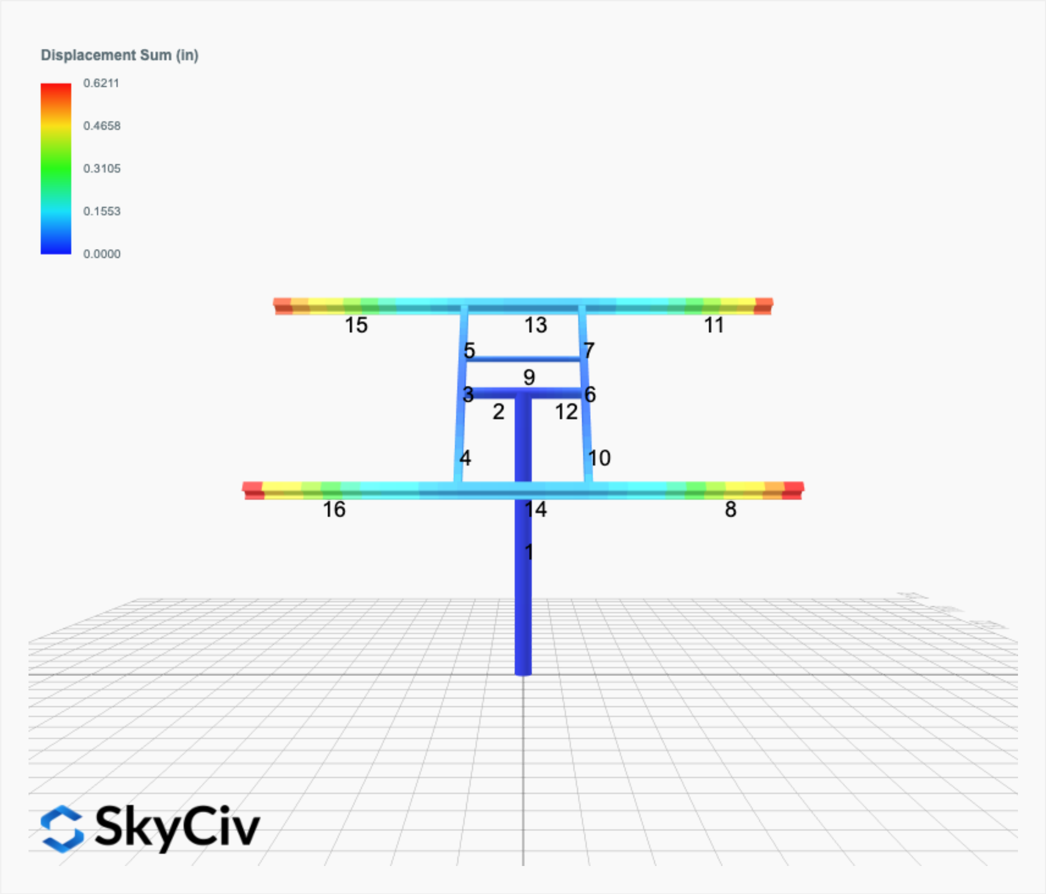
Rail Length: 13.666666666666666 ft
Additional Restraints Required: None
Tributary Width: 3.1 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0411 kip/ft
Snow (Y): -0.0586 kip/ft
Wind uplift Case A: 0.0509 kip/ft
Wind downforce Case A: 0.0509 kip/ft
Dead (Panel load) (X): 0.0084 kip/ft
Dead (Panel load) (Y): -0.0120 kip/ft

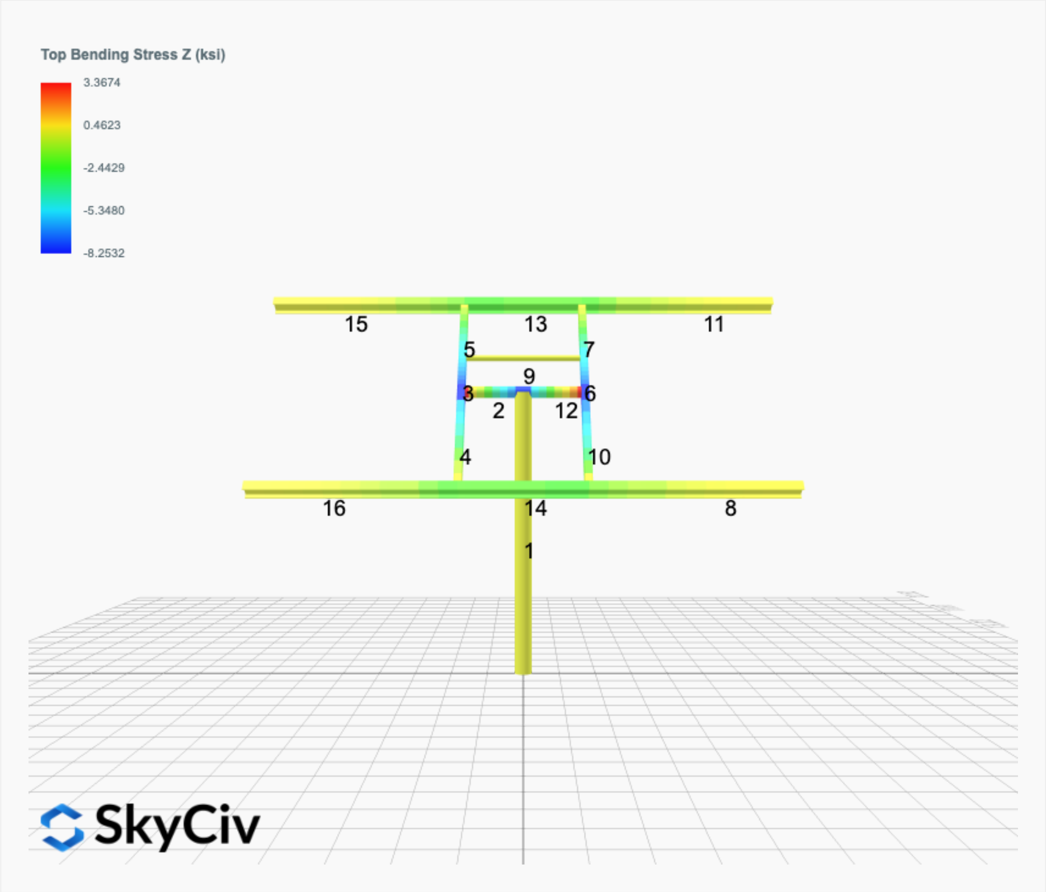


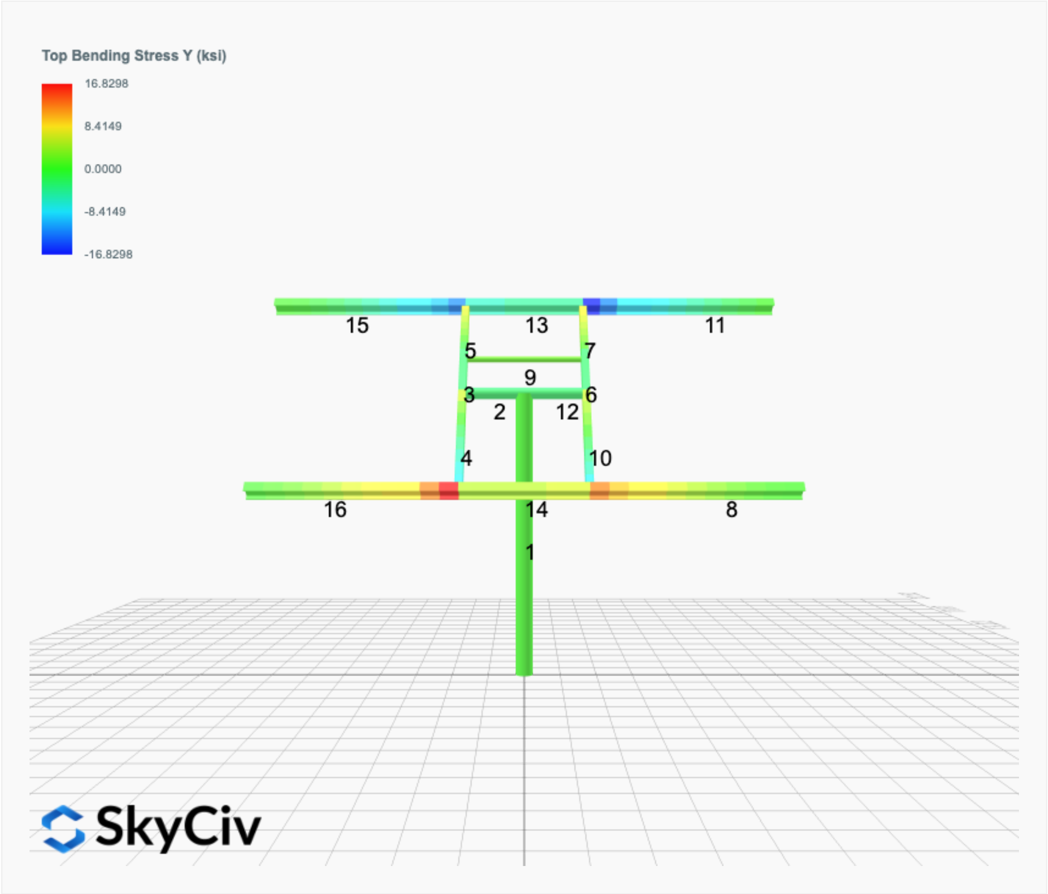
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	14.03808752	0.407	PASS
Material Yield	34.5	14.03808752	0.407	PASS
Material Strength	37	14.03808752	0.379	PASS



FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
16.8298
8.4149
0.0000
-8.4149
-16.8298
15
13
11
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3
6
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12
4
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16
14
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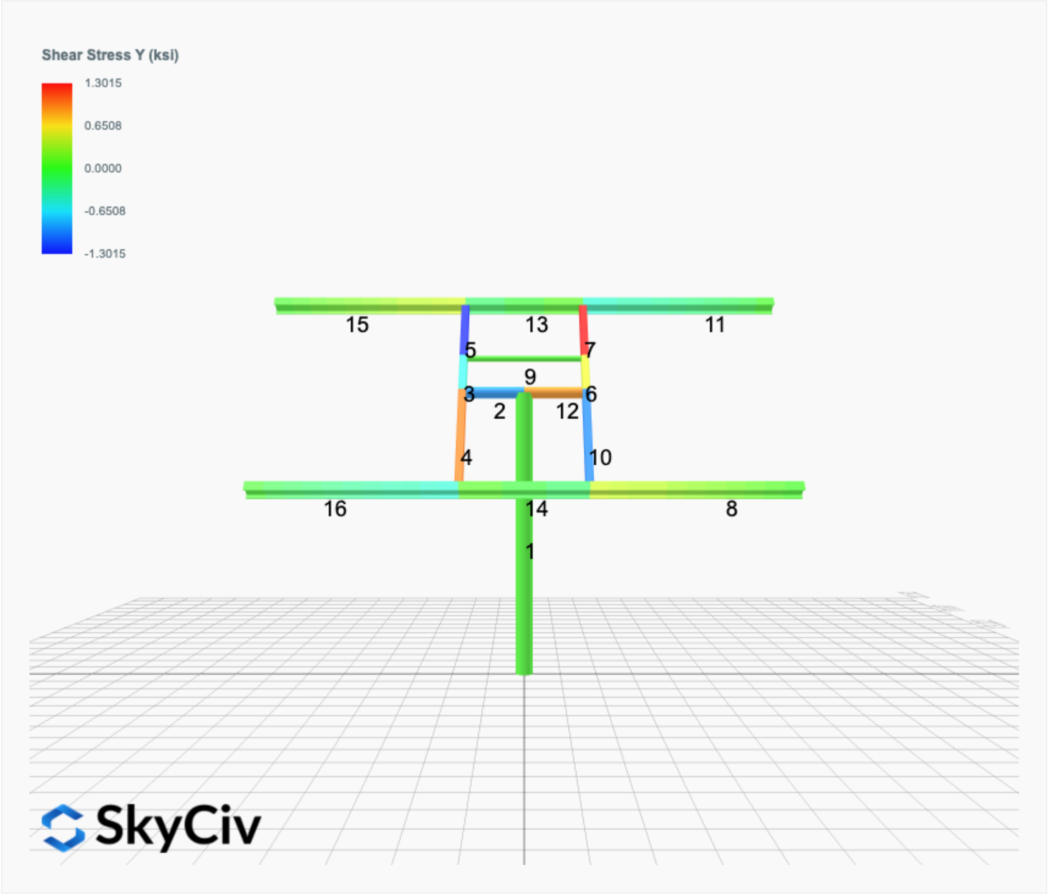
1
SkyCiv


Shear Stress Y (ksi)
1.3015
0.6508
0.0000
-0.6508
-1.3015
15
13
11
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4
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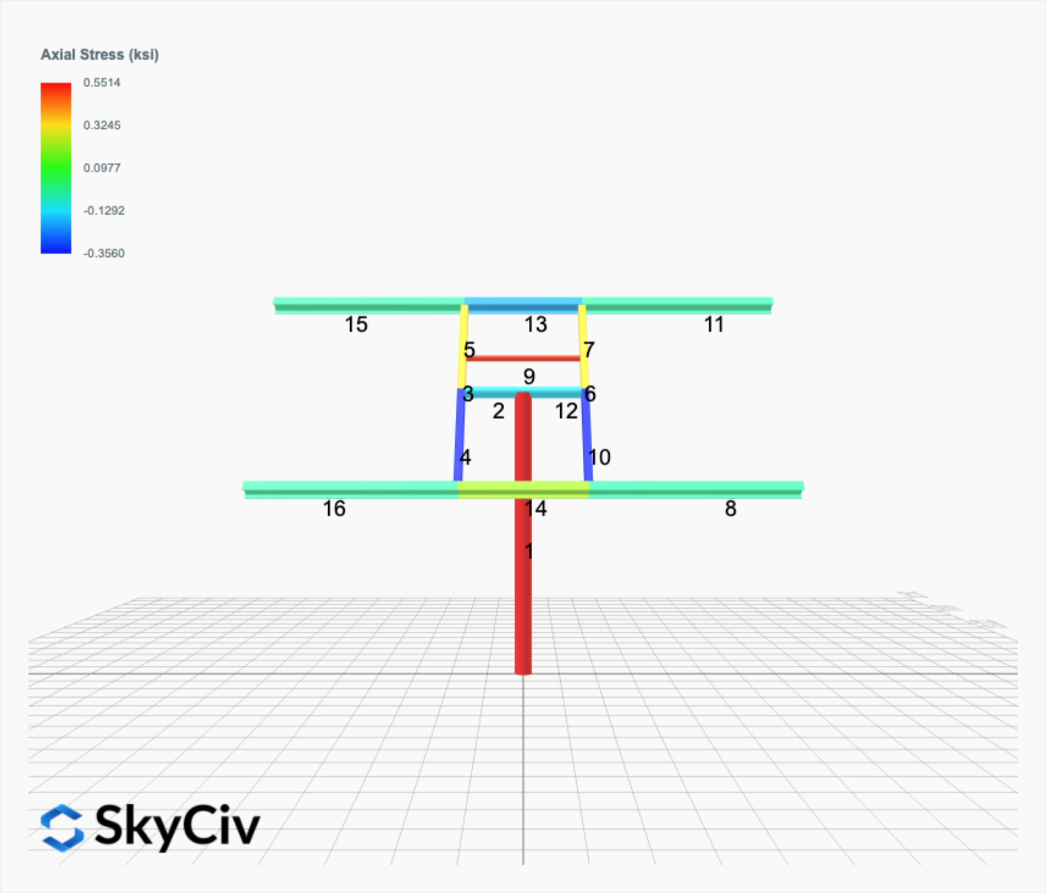
SkyCiv



Axial Stress (ksi)
0.5514
0.3245
0.0977
-0.1292
-0.3560
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	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 2. D + L	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 3. D + (S or Lr or R)	0.0000	4.9051	0.0000	0.0000	-0.0000	0.0311
ULS: 3. D + (S or Lr or R)	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.1358	0.0000	0.0000	-0.0000	0.0270
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 5b. D + 0.7E	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.1358	0.0000	0.0000	-0.0000	0.0270
ULS: 8. 0.6D + 0.7E	0.0000	1.0967	0.0000	0.0000	-0.0000	0.0088
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0508	3.2638	0.0000	0.0000	-0.0000	18.8922
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0508	0.3918	0.0000	0.0000	-0.0000	-18.4282
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5381	5.2128	0.0000	0.0000	-0.0000	14.1852
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.1358	0.0000	0.0000	-0.0000	0.0270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5381	3.0588	0.0000	0.0000	-0.0000	-13.8051
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.1358	0.0000	0.0000	-0.0000	0.0270
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5381	2.9048	0.0000	0.0000	-0.0000	14.1728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5381	0.7508	0.0000	0.0000	-0.0000	-13.8175
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.8278	0.0000	0.0000	-0.0000	0.0146
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0508	2.5327	0.0000	0.0000	-0.0000	18.8864
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.0967	0.0000	0.0000	-0.0000	0.0088
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0508	-0.3393	0.0000	0.0000	-0.0000	-18.4340
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.0967	0.0000	0.0000	-0.0000	0.0088

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	8.3137
Shear X	-3.4180
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	32.1376

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	5.2128
Shear X	-2.0508
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	18.8922

Project Details

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

Design Input Information

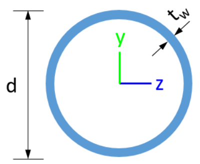
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

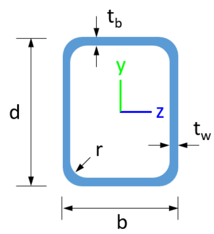
Design Materials

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

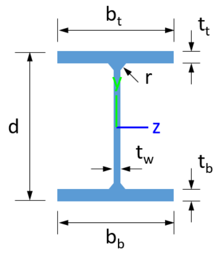
Section Dimensions



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	19.10	19.10	9.10	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
8	18	9.97	9.97	4.75	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	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	9.97	9.97	4.75	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	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
14	18	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
15	18	9.97	9.97	4.75	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	9.97	9.97	4.75	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	117.21	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	34.69	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	34.69	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	18.15	6.45	30.09	45.74
14	120.60	84.03	18.13	6.45	30.09	45.74

15	120.60	34.69	23.36	6.45	30.09	45.74
16	120.60	34.69	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	0.071	0.760	0.000	0.045	0.000	0.786	#13	0.510	Not Required	Pass
2	0.007	0.370	0.228	0.095	0.040	0.535	#21	0.034	Not Required	Pass
3	0.019	0.531	0.114	0.053	0.006	0.643	#21	0.044	Not Required	Pass
4	0.018	0.527	0.383	0.053	0.063	0.768	#21	0.078	Not Required	Pass
5	0.019	0.329	0.401	0.053	0.081	0.430	#21	0.073	Not Required	Pass
6	0.019	0.531	0.114	0.053	0.006	0.643	#21	0.044	Not Required	Pass
7	0.019	0.329	0.401	0.053	0.081	0.431	#21	0.073	Not Required	Pass
8	0.000	0.087	0.284	0.028	0.017	0.368	#21	Not Required	Not Required	Pass
9	0.029	0.030	0.063	0.001	0.000	0.098	#21	0.198	Not Required	Pass
10	0.018	0.527	0.383	0.053	0.063	0.768	#21	0.078	Not Required	Pass
11	0.000	0.087	0.284	0.028	0.017	0.368	#21	Not Required	Not Required	Pass
12	0.007	0.370	0.228	0.095	0.040	0.535	#21	0.034	Not Required	Pass
13	0.011	0.214	0.531	0.039	0.023	0.735	#21	0.177	Not Required	Pass
14	0.012	0.218	0.531	0.039	0.023	0.735	#21	0.177	Not Required	Pass
15	0.000	0.087	0.284	0.028	0.017	0.368	#21	Not Required	Not Required	Pass
16	0.000	0.087	0.284	0.028	0.017	0.368	#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

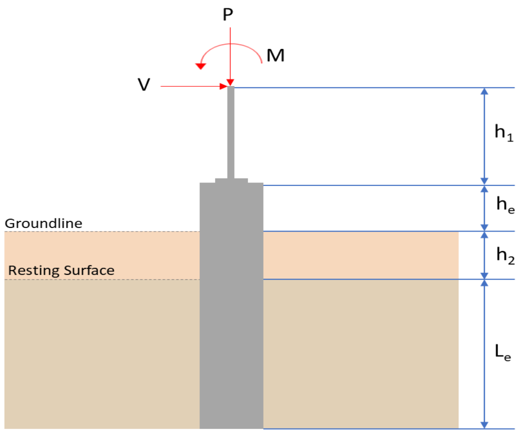
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 60 \text{ in}$ - Pile width
$D = 60 \text{ in}$ - Pile depth
$L = 5.25 \text{ ft}$ - Total pile length
$h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile,
$h_2 = 0 \text{ ft}$ - Depth to resisting surface
$h_e = 0 \text{ ft}$ - 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)	5.213	8.314
V_x (kip)	-2.051	-3.418
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	18.892	32.138

Material Properties

$f'_{ck} = 2.5 \text{ ksi}$ - 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,

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

$$H_o = \frac{(-2.051 \text{ kip})}{1.57 \times (60 \text{ in})}$$

$$H_o = -0.26127 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.892\ \mathrm{kipft}) + ((-2.051\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (60\ \mathrm{in})}$$

$$M_o = 2.4066\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 4.8779\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

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

$$L_{e,req} = MAX[(4.8779\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 4.878\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.878\ \mathrm{ft})}{(5.25\ \mathrm{ft})}$$

$$Ratio = 0.92914$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\ D$$

$$A = (60\ \mathrm{in}) \times (60\ \mathrm{in})$$

$$A = 25\ \mathrm{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.213\ \mathrm{kip})}{(25\ \mathrm{ft}^2)}$$

$$q = 0.20852\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.20852\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.10426$$

Status: **PASS**
Ratio: **0.100**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25\ \mathrm{ft})}{(60\ \mathrm{in})}$$

$$L/D = 1.05$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.26127\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.4066\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4066\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.26127\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (2.4066\ \mathrm{kipft/ft})) + (4 \times (-0.26127\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.6205\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.4066\ \mathrm{kipft/ft})) + (3 \times (-0.26127\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{\ 2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (2.4066\ \mathrm{kipft/ft})) + (2 \times (-0.26127\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = 0.18464\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.4066\ \mathrm{kipft/ft})) + ((-0.26127\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.74918\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.6205\ \mathrm{ft})}{2}$$

$$p_a = 0.27153\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.18464\ \mathrm{kip/ft^2}\right)}{\left(0.27153\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.68$$

Status: **PASS**
Ratio: **0.680**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

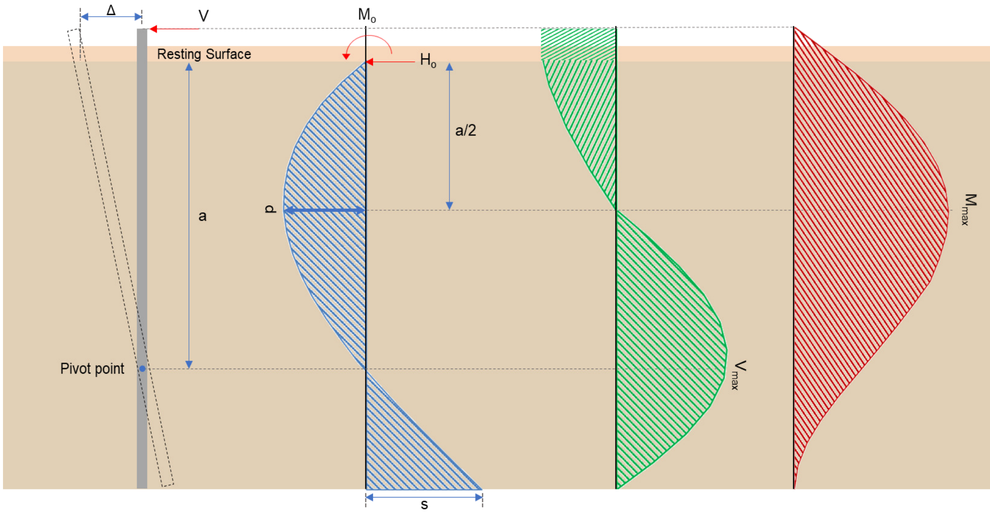
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.74918\ \mathrm{kip/ft^2}\right)}{\left(0.7875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.95134$$

Status: PASS
Ratio: 0.950



Pile Rotation Pressure Diagram Shear Diagram Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.418\ \text{kip})}{1.57 \times (60\ \text{in})}$$

$$H_o = -0.43541\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.138\ \text{kipft}) + ((-3.418\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (60\ \text{in})}$$

$$M_o = 4.094\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.094\ \text{kipft/ft})}{(-0.43541\ \text{kip/ft})}$$

$$E = 9.4026\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.094\ \text{kipft/ft}) \times (5.25\ \text{ft})) + \left(3 \times (-0.43541\ \text{kip/ft}) \times (5.25\ \text{ft})^2\right)}{(6 \times (4.094\ \text{kipft/ft})) + (4 \times (-0.43541\ \text{kip/ft}) \times (5.25\ \text{ft}))}$$

$$a = 3.6187\ \text{ft}$$

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

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

$$V_{max} = ((-0.43541\ \text{kip/ft}) \times (60\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.4026\ \text{ft})}{(5.25\ \text{ft})} + 3\right) \times \left(\frac{(3.6187\ \text{ft})}{(5.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.4026\ \text{ft})}{(5.25\ \text{ft})} + 2\right) \times \left(\frac{(3.6187\ \text{ft})}{(5.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.3356\ \text{kip}$$

$$V_{max} = 8.3356 \text{ kip}$$

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

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

$$M_{max} = ((-0.43541 \text{ kip/ft}) \times (60 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(9.4026 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6187 \text{ ft})}{2 \times (5.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (9.4026 \text{ ft})}{(5.25 \text{ ft})} + 3\right) \times \left(\frac{(3.6187 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (9.4026 \text{ ft})}{(5.25 \text{ ft})} + 2\right) \times \left(\frac{(3.6187 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 20.843 \text{ kipft}$$

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,
$A_g = 3600 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(8.314 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (3600 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (3600 \text{ in}^2))\right]$$

$$A_{st,required} = -131.91 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-131.91 \text{ in}^2), (0.0018 \times (3600 \text{ in}^2))\right]$$

$$A_{min} = 6.48 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(6.48 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 22$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\, \frac{\pi\, d_{bar}^2}{4}$$

$$A_{st} = (22) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 6.7495 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.48 \text{ in}^2)}{(6.7495 \text{ in}^2)}$$

$$Ratio = 0.96007$$

Status: **PASS**
Ratio: **0.960**

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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,

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

$$s_{ties} = Min\left[(16 \times (0.625\ \text{in})), (48 \times (0.375\ \text{in})), Min\,((60\ \text{in}), (60\ \text{in}))\right]$$

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

Summary:

Main reinforcement: **22 - #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

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

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(3600\ \text{in}^2\right) - \left(6.7495\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(6.7495\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 4181.1\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.314\ \text{kip})}{(4181.1\ \text{kip})}$$

$$Ratio = 0.0019885$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 60\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (60\ \text{in})$$

$$d = 48\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(48\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.58722$$

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

$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$

$$V_{c,max} = 5 \times (0.58722) \times \sqrt{(2500\ \text{psi})} \times (60\ \text{in}) \times (48\ \text{in})$$

$$V_{c,max} = 422.8\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 8.314\ \text{kip} \rightarrow 8314\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + \frac{(8314 \text{ lbf})}{6 \times (3600 \text{ in}^2)}\right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,a} = 170.23 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.58722) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{c,b} = 529.12 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(422.8 \text{ kip}), (170.23 \text{ kip}), (529.12 \text{ kip})\right]$$

$$V_c = 170.23 \text{ kip}$$

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)

$$V_{s,a} = 8\ \sqrt{f'_{ck}}\ b_w\ d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (60 \text{ in}) \times (48 \text{ in})$$

$$V_{s,a} = 1152 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (48 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 63.617 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(1152 \text{ kip}), (63.617 \text{ kip})\right]$$

$$V_s = 63.617 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((170.23 \text{ kip}) + (63.617 \text{ kip}))$$

$$\phi V_n = 152 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.3356 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(8.3356 \text{ kip})}{(152 \text{ kip})}$$

$$Ratio = 0.05484$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b\,D^2}{6}$$

$$S_m = \frac{(60\text{ in}) \times (60\text{ in})^2}{6}$$

$$S_m = 36000\text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 35999.999in^3$$

$$\phi M_{n,1} = 487.500kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\; 0.85\; f_{ck}'\; S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\text{ ksi}) \times \left(36000\text{ in}^3\right)$$

$$\phi M_{n,2} = 4143.7\text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(487.5\text{ kipft}), (4143.7\text{ kipft})]$$

$$\phi M_n = 487.5\text{ kipft}$$

Considering x-direction:

$M_{max} = 20.843\text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(20.843\text{ kipft})}{(487.5\text{ kipft})}$$

$$Ratio = 0.042754$$

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