Project Details

Project Name: TownofCG-TownHall Example - V1Jb

Location: 4058 County Hwy N, Cottage Grove, WI 53527, USA

Unique ID: 1P-0-6TOP-SD-84-L-4Hx4W-STRUTS-L18J

Dealer: ______________________________

Date: Fri Apr 04 2025

Number of Modules: 16

Number of Poles: 1

Date Sold:

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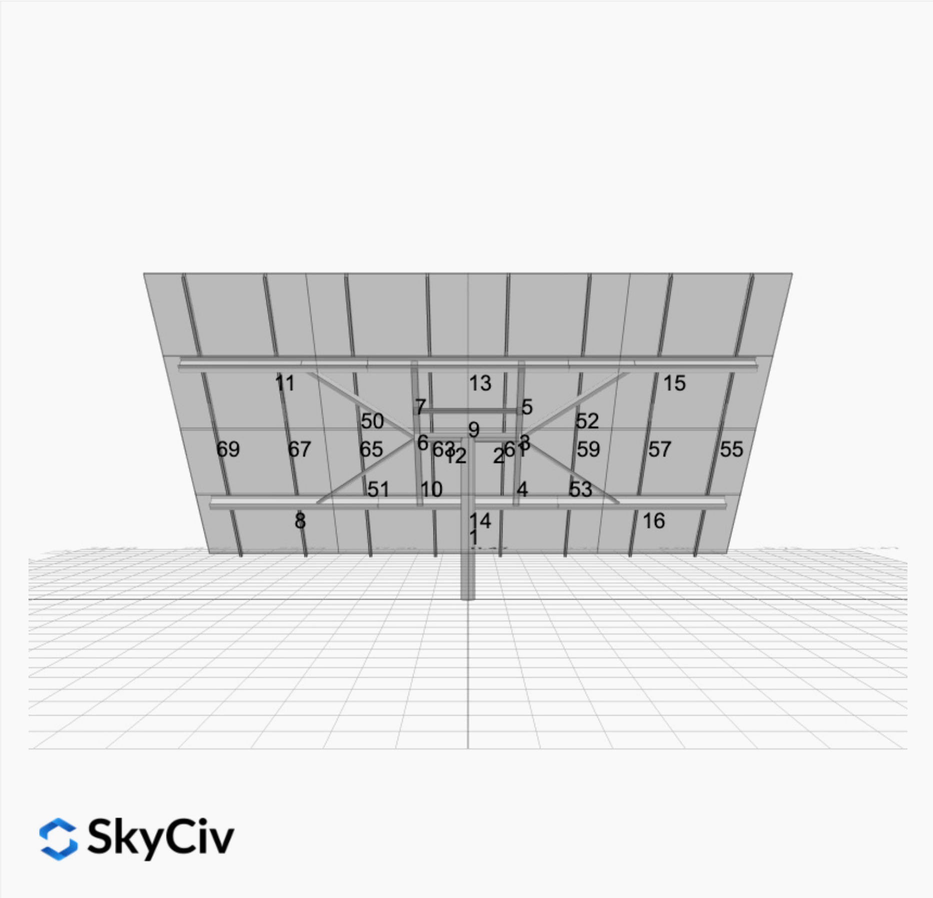


Array Dimensions N/S	15.03 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	46
Front Edge Clearance	1 ft

MT Solar Bill of Materials (1P-0-6TOP-SD-84-L-4Hx4W-STRUTS-L18J)

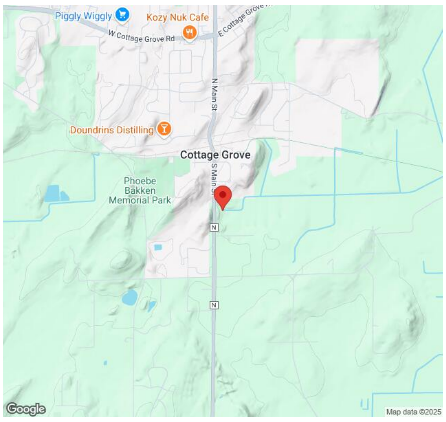
Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-84	84IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	4

Rail Bill of Materials

Part	Qty
Rails (180in)	8
Rail Attachment	16
Module Mid Clamp	24
Module End Clamp	16

Part	Qty
Ground Lug	4

Site Details:



Site Address: 4058 County Hwy N, Cottage Grove, WI 53527, USA

Array Specification

Duty Classification:	SD
Module Width:	44.60 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Winter Tilt Angle:	46
Front Edge Clearance:	1
Total Array Height at Tilt:	11.81 ft
Total Frame Length:	21.50 ft
Module Info/Notes:	SIL-430 QD
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	22.93 ft
Rail Length:	180.40 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	6.41 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	72 x 72 in
Foundation Depth (below grade):	Pile 1: 4.75 ft
Foundation Volume:	6.333 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	4058 County Hwy N, Cottage Grove, WI 53527, USA
Wind Speed:	100 mph
Snow Load:	30 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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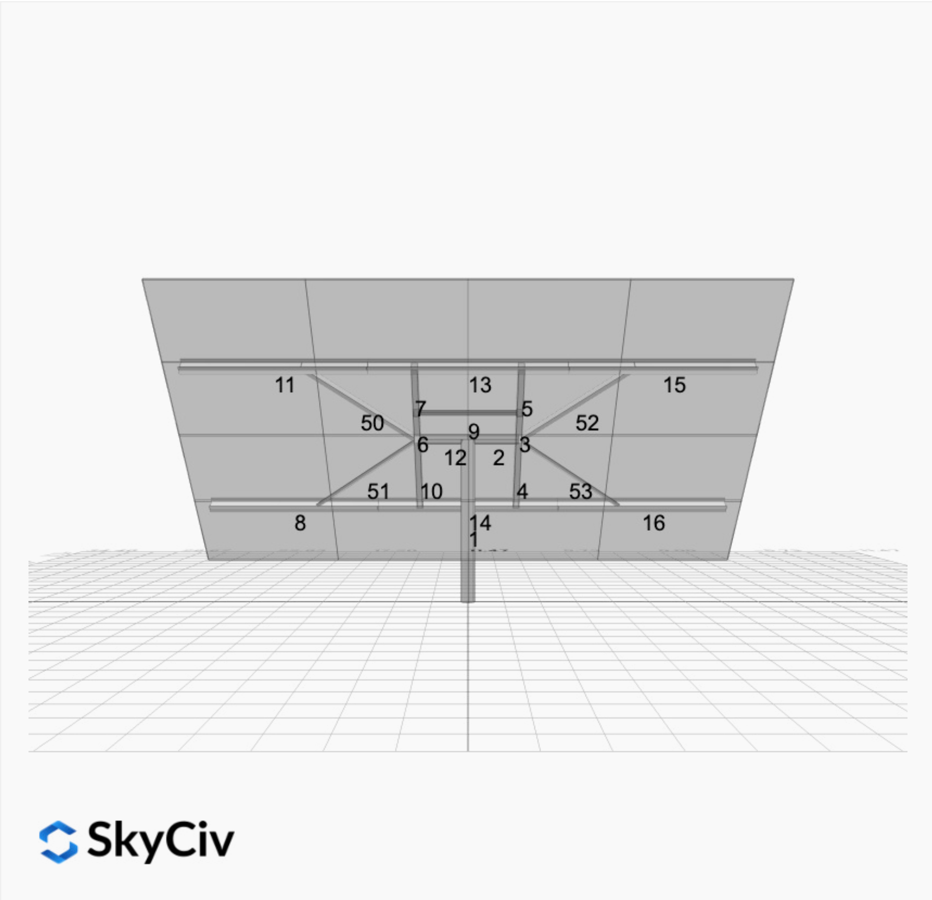
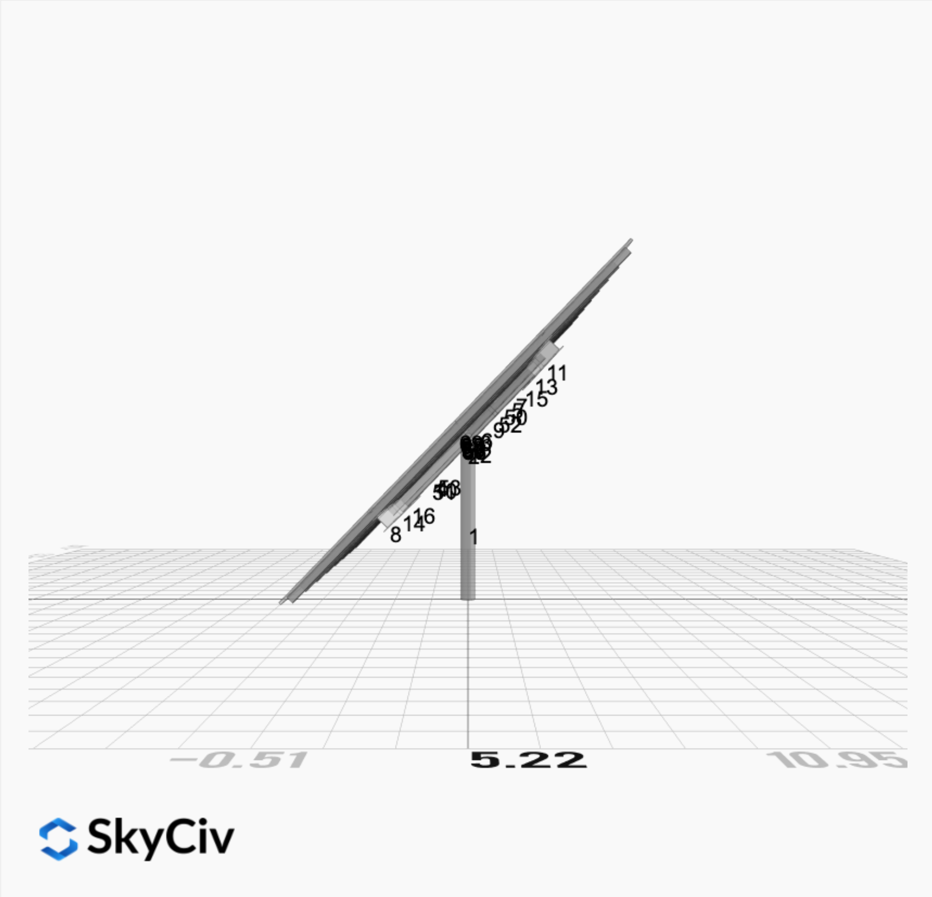
Design Notes:

- 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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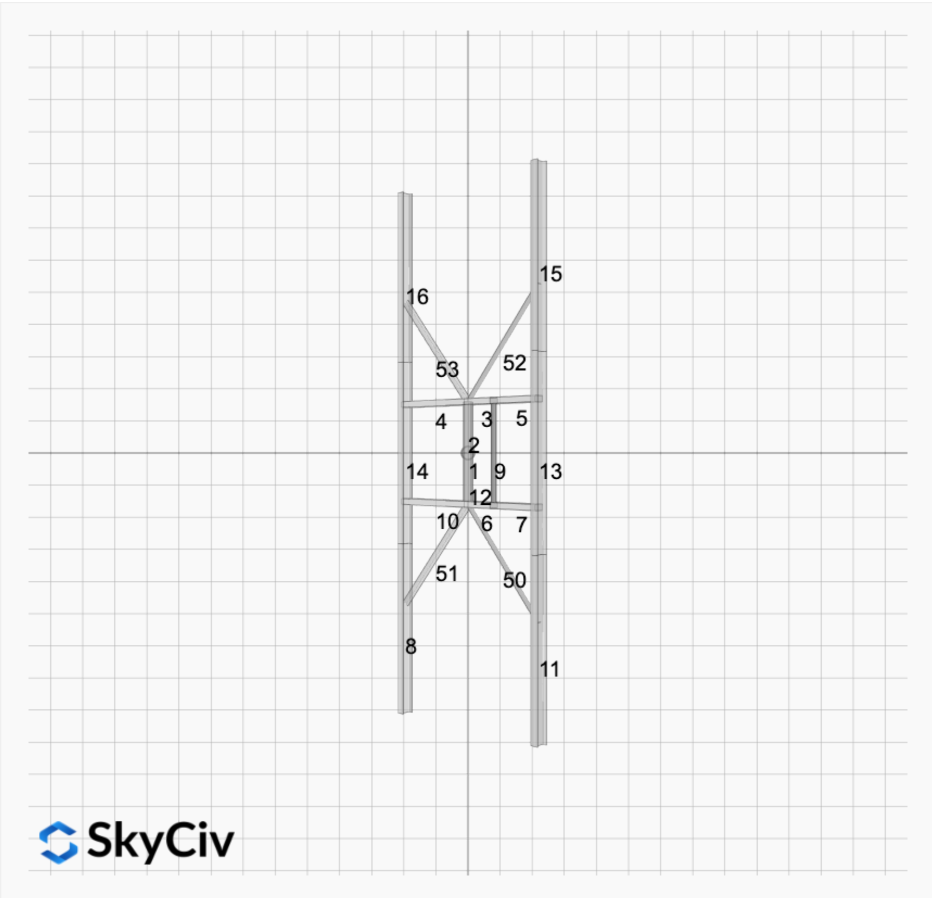
13
15
8
14
16
1
-0.51
5.22
10.95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12
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6
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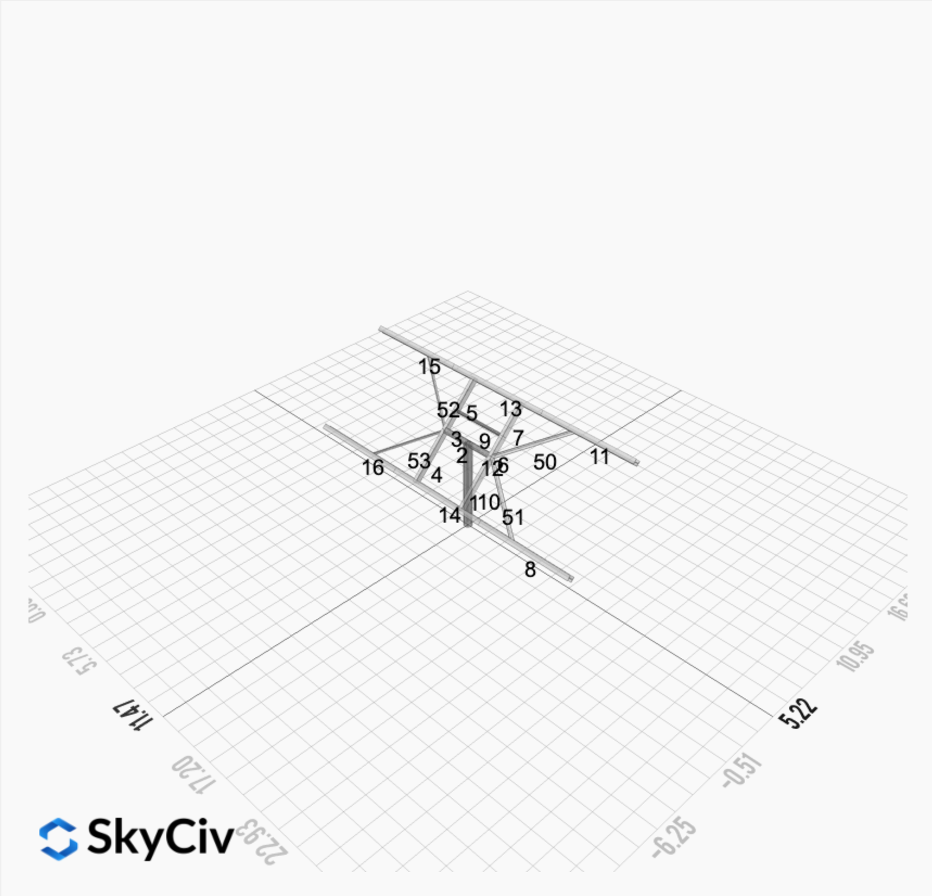
8
11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5
13
3
9
7
16
53
2
12
6
50
11
4
14
110
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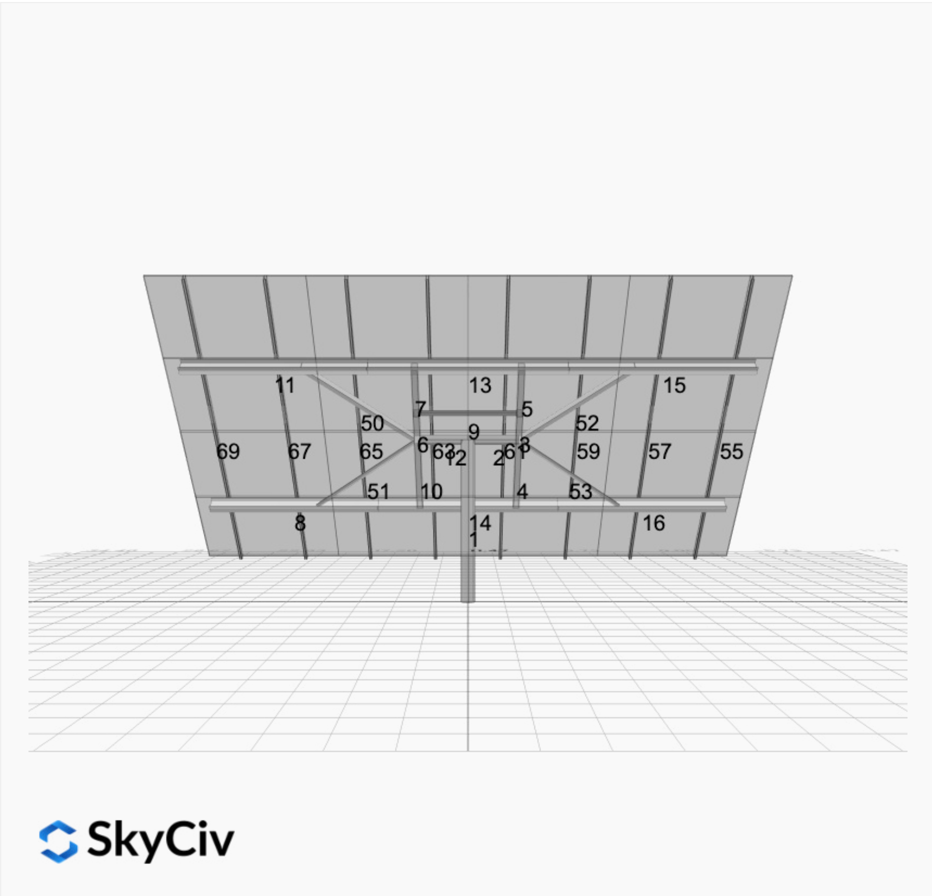
5.73
11.47
17.20
22.93
5.22
10.95
-0.51
-6.25
SkyCiv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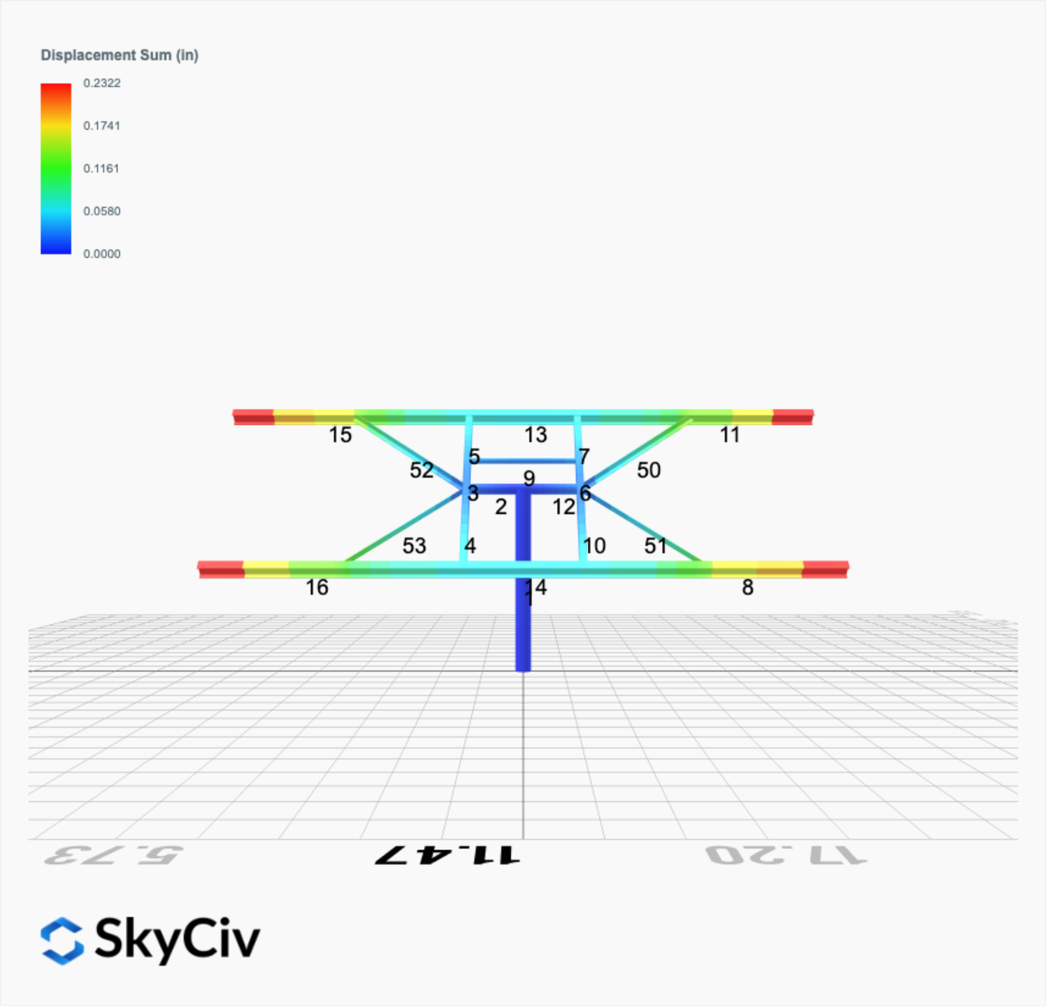
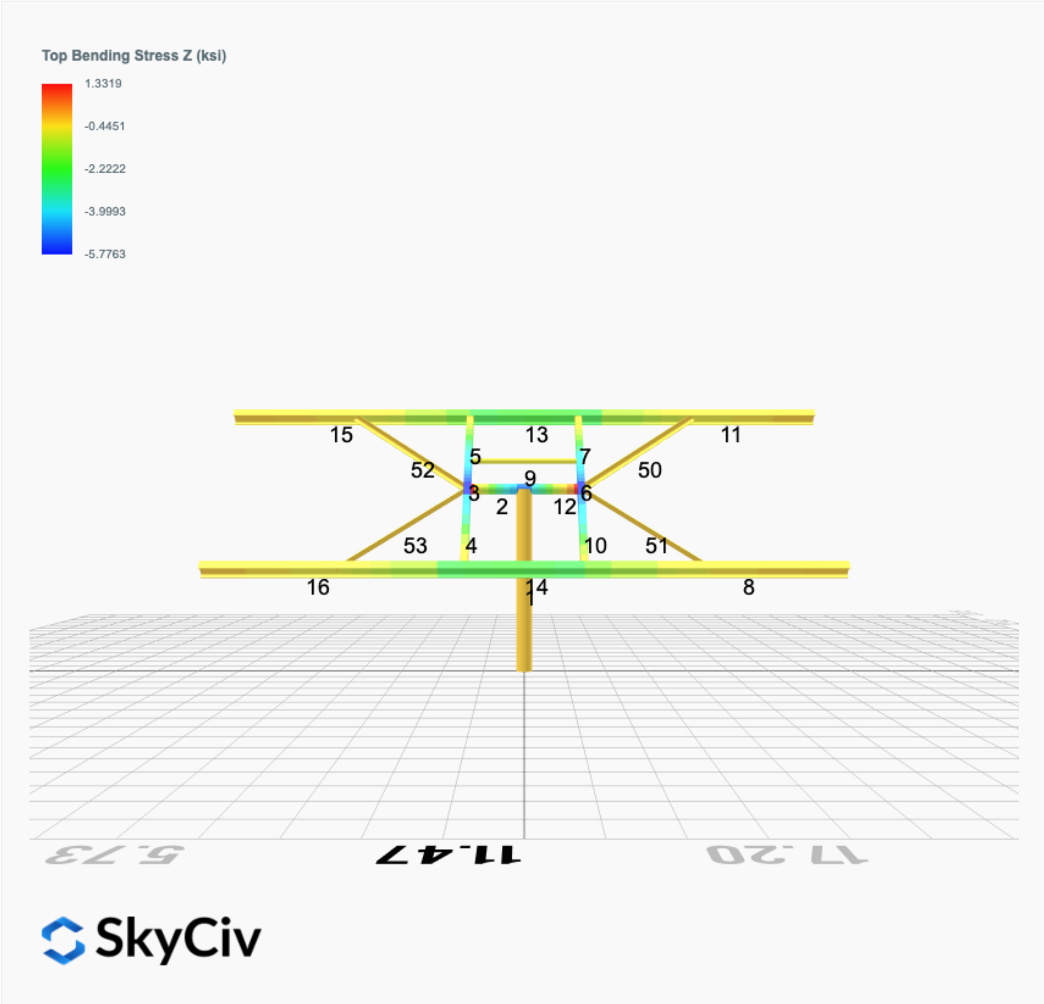
13
15
7
5
50
52
9
69
67
65
6
63
12
2
61
8
59
57
55
51
10
4
53
8
14
1
16
SkyCiv

FEM Results (Envelope Worst Case for each member)

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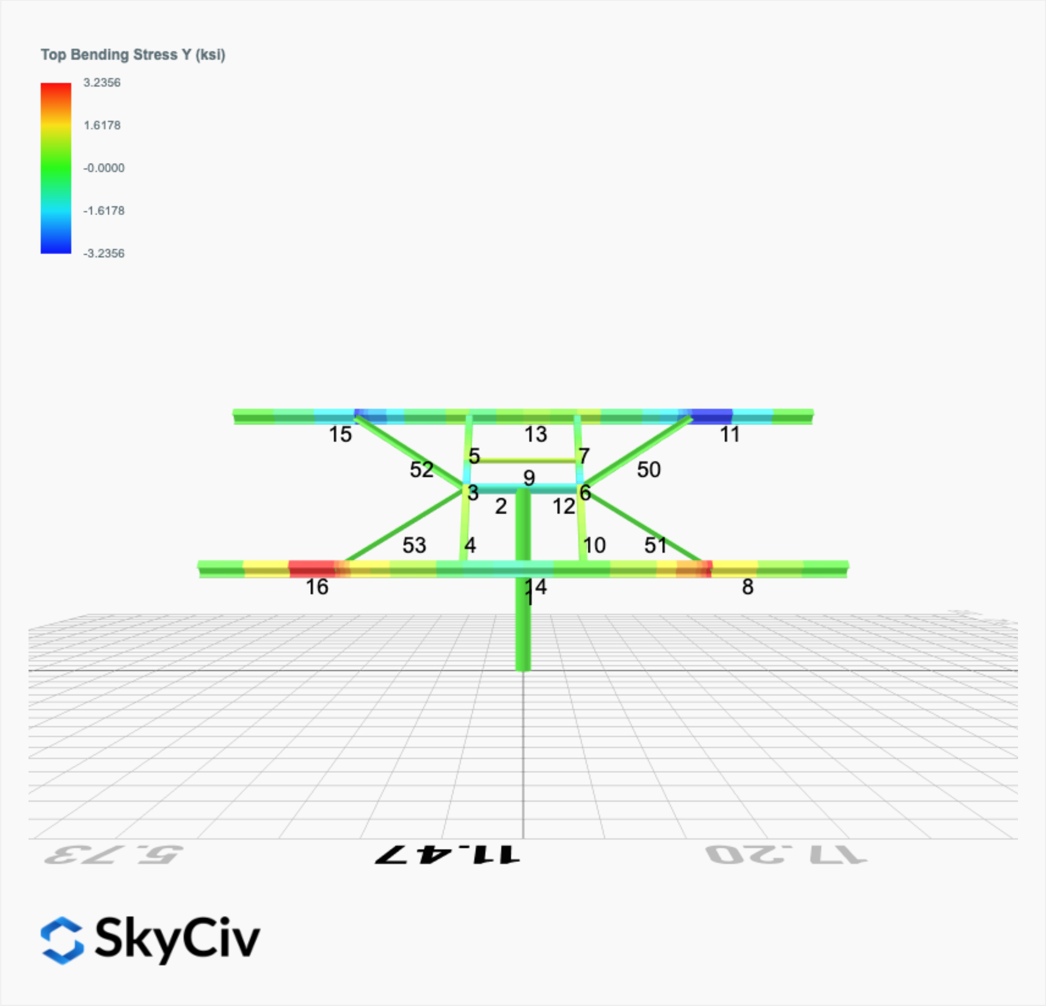
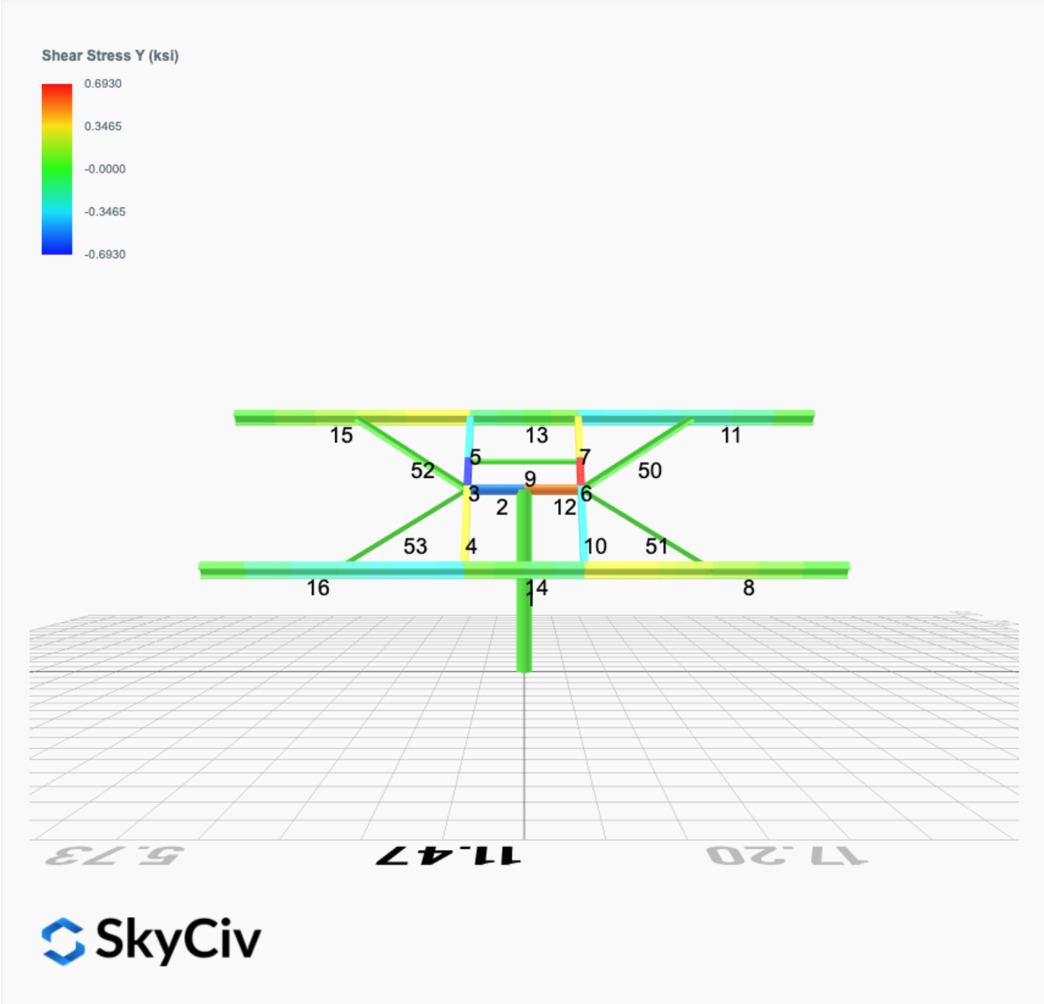
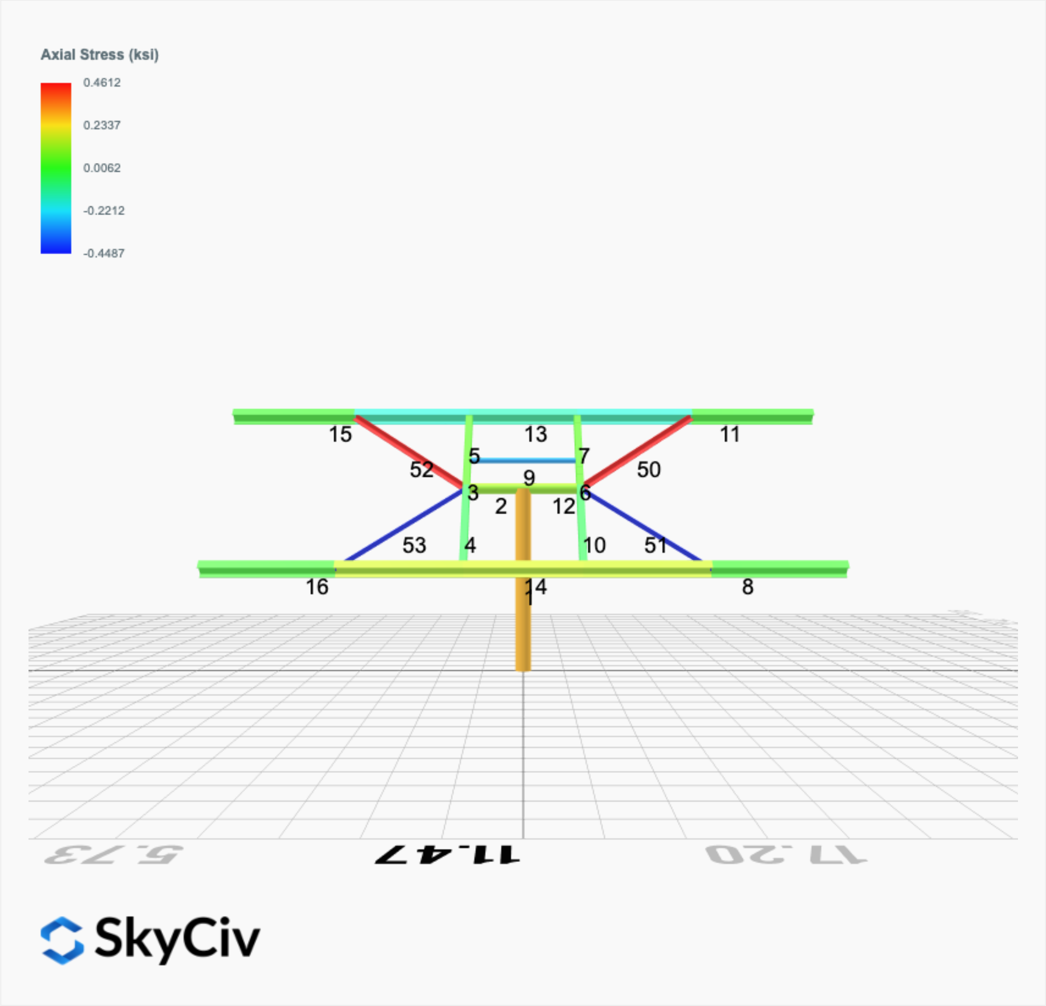
1.3319
-0.4451
-2.2222
-3.9993
-5.7763
15
13
11
5
7
52
9
50
3
6
2
12
53
4
10
51
16
14
1
8
5.73
11.47
17.20
SkyCiv


Top Bending Stress Y (ksi)
3.2356
1.6178
-0.0000
-1.6178
-3.2356
15
13
11
52
5
7
50
9
3
6
2
12
53
4
10
51
16
14
1
8
5.73
11.47
17.20



Shear Stress Y (ksi)
0.6930
0.3465
-0.0000
-0.3465
-0.6930
15
13
11
5
7
52
9
50
3
6
2
12
53
4
10
51
16
14
1
8
SkyCiv


Axial Stress (ksi)
0.4612
0.2337
0.0062
-0.2212
-0.4487
15
13
11
52
5
7
50
3
9
6
2
12
53
4
10
51
16
14
1
8
SkyCiv

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	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 2. D + L	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 3. D + (S or Lr or R)	0.0000	4.1295	0.0000	-0.0000	-0.0000	0.0358
ULS: 3. D + (S or Lr or R)	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.6851	0.0000	-0.0000	-0.0000	0.0330
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 5b. D + 0.7E	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.6851	0.0000	-0.0000	-0.0000	0.0330
ULS: 8. 0.6D + 0.7E	0.0000	1.4111	0.0000	-0.0000	-0.0000	0.0147
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6073	4.8696	0.0000	-0.0000	-0.0000	16.9007
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6073	-0.1660	0.0000	-0.0000	-0.0000	-16.5132
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9555	5.5734	0.0000	-0.0000	-0.0000	12.6901
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.6851	0.0000	-0.0000	-0.0000	0.0330
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9555	1.7967	0.0000	-0.0000	-0.0000	-12.3703
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.6851	0.0000	-0.0000	-0.0000	0.0330
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9555	4.2402	0.0000	-0.0000	-0.0000	12.6817
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9555	0.4635	0.0000	-0.0000	-0.0000	-12.3788
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.3518	0.0000	-0.0000	-0.0000	0.0245
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6073	3.9289	0.0000	-0.0000	-0.0000	16.8909
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.4111	0.0000	-0.0000	-0.0000	0.0147
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6073	-1.1067	0.0000	-0.0000	-0.0000	-16.5230
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.4111	0.0000	-0.0000	-0.0000	0.0147

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	7.9073
Shear X	-4.3455
Shear Z	0.0000
Moment X	-0.0012
Moment Y (Twist)	0.0004
Moment Z	28.5529

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	5.5734
Shear X	-2.6073
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	16.9007

Project Details

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

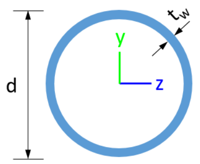
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

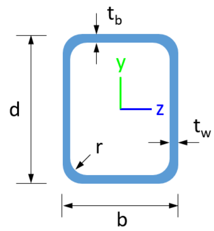
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

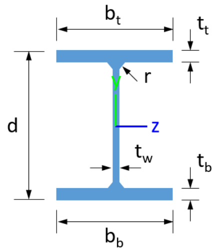
Section Dimensions



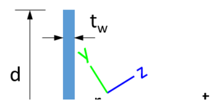
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
7	6in Pipe Sch 40	6.63	0.28					

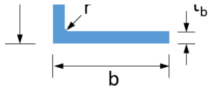


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		



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





ID	Name	d (in)	t_w (in)	b (in)	t_b (in)	r (in)		
34	L3x2x3/16	3.00	0.19	2.00	0.19	0.31		

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
34	L3x2x3/16	0.92	0.01	0.17	0.98	0.01	0.33	0.82

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	13.45	13.45	6.41	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
8	18	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.29,2.31,2.30,2.30,2.30,2.30,2.30,2.31,2.28,2.31,2.30,2.31,2.30,2.31	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	7.00	7.00	7.00	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.30,2.31,2.31,2.32,2.29,2.32,2.30,2.31,2.30,2.31	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.02,1.02,1.02,1.02,1.02	300	200	1
14	18	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	18	7.00	7.00	7.00	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.31,2.31,2.31,2.32,2.29,2.32,2.30,2.31,2.30,2.31	300	200	1
16	18	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.29,2.31,2.30,2.30,2.30,2.30,2.30,2.31,2.28,2.31,2.30,2.31,2.30,2.31	300	200	1
50	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
51	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
52	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
53	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250

Member Design Capacity

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	172.10	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	64.33	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	64.33	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	17.94	6.45	30.09	45.74
14	120.60	84.03	17.93	6.45	30.09	45.74
15	120.60	64.33	23.36	6.45	30.09	45.74
16	120.60	64.33	23.36	6.45	30.09	45.74
50	41.27	8.45	1.63	0.76	15.23	10.15
51	41.27	8.45	1.63	0.76	15.23	10.15
52	41.27	8.45	1.63	0.76	15.23	10.15
53	41.27	8.45	1.63	0.76	15.23	10.15

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	0.046	0.675	0.000	0.058	0.000	0.698	#13	0.360	Not Required	Pass
2	0.006	0.423	0.289	0.091	0.051	0.714	#13	0.052	Not Required	Pass
3	0.002	0.725	0.164	0.073	0.053	0.831	#13	0.044	Not Required	Pass
4	0.002	0.717	0.054	0.072	0.004	0.753	#13	0.078	Not Required	Pass
5	0.002	0.450	0.037	0.073	0.012	0.473	#13	0.073	Not Required	Pass
6	0.002	0.725	0.164	0.073	0.053	0.831	#13	0.044	Not Required	Pass
7	0.002	0.450	0.037	0.073	0.012	0.474	#13	0.073	Not Required	Pass
8	0.014	0.205	0.138	0.045	0.009	0.223	#13	0.464	Not Required	Pass
9	0.022	0.073	0.077	0.001	0.000	0.157	#13	0.132	Not Required	Pass
10	0.002	0.717	0.054	0.072	0.004	0.753	#13	0.078	Not Required	Pass
11	0.008	0.205	0.138	0.046	0.010	0.216	#13	0.309	Not Required	Pass
12	0.006	0.423	0.289	0.091	0.051	0.714	#13	0.052	Not Required	Pass
13	0.008	0.418	0.023	0.057	0.005	0.435	#13	0.177	Not Required	Pass
14	0.012	0.422	0.028	0.057	0.004	0.426	#13	0.265	Not Required	Pass
15	0.008	0.205	0.138	0.046	0.010	0.216	#13	0.309	Not Required	Pass
16	0.014	0.205	0.138	0.045	0.009	0.223	#13	0.309	Not Required	Pass
50	0.150	0.009	0.005	0.002	0.001	0.162	#21	0.783	Not Required	Pass
51	0.030	0.005	0.016	0.001	0.001	0.047	#6	0.522	Not Required	Pass
52	0.150	0.009	0.005	0.002	0.001	0.162	#23	0.783	Not Required	Pass
53	0.030	0.005	0.016	0.001	0.001	0.047	#6	0.522	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

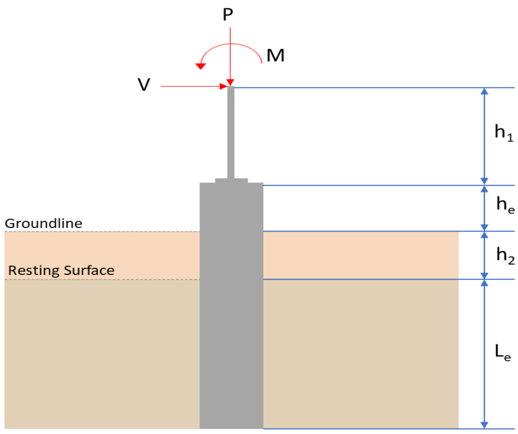
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 72\ \mathrm{in}$ - Pile width

$D = 72\ \mathrm{in}$ - Pile depth

$L = 4.75\ \mathrm{ft}$ - Total pile length

$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,

$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface

$h_e = 0\ \mathrm{ft}$ - 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)	5.573	7.907
V_x (kip)	-2.607	-4.345
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	16.901	28.553

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.607\ \mathrm{kip})}{1.57 \times (72\ \mathrm{in})}$$

$$H_o = -0.27675\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.901\ \text{kipft}) + ((-2.607\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (72\ \text{in})}$$

$$M_o = 1.7942\ \text{kipft/ft}$$

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:
$L_{e,x} = 4.1957\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(4.1957\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 4.196\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 4.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.196\ \text{ft})}{(4.75\ \text{ft})}$$

$$Ratio = 0.88337$$

Status: **PASS**
Ratio: **0.880**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (72\ \text{in}) \times (72\ \text{in})$$

$$A = 36\ \text{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.573\ \text{kip})}{(36\ \text{ft}^2)}$$

$$q = 0.15481\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.15481\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.077403$$

Status: **PASS**
Ratio: **0.080**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.75\ \mathrm{ft})}{(72\ \mathrm{in})}$$

$$L/D = 0.79167$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.27675\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 1.7942\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7942\ \mathrm{kipft/ft}) \times (4.75\ \mathrm{ft})) + \left(3 \times (-0.27675\ \mathrm{kip/ft}) \times (4.75\ \mathrm{ft})^2\right)}{(6 \times (1.7942\ \mathrm{kipft/ft})) + (4 \times (-0.27675\ \mathrm{kip/ft}) \times (4.75\ \mathrm{ft}))}$$

$$a = 3.2966\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7942\ \mathrm{kipft/ft})) + (3 \times (-0.27675\ \mathrm{kip/ft}) \times (4.75\ \mathrm{ft}))]^{\ 2}}{(4.75\ \mathrm{ft})^2 \times [(3 \times (1.7942\ \mathrm{kipft/ft})) + (2 \times (-0.27675\ \mathrm{kip/ft}) \times (4.75\ \mathrm{ft}))]}$$

$$p = 0.12618\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (1.7942\ \mathrm{kipft/ft})) + ((-0.27675\ \mathrm{kip/ft}) \times (4.75\ \mathrm{ft}))]}{(4.75\ \mathrm{ft})^2}$$

$$s = 0.60465\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.2966\ \mathrm{ft})}{2}$$

$$p_a = 0.24724\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.12618\ \mathrm{kip/ft^2}\right)}{\left(0.24724\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.51037$$

Status: **PASS**
Ratio: **0.510**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (4.75\ \mathrm{ft})$$

$$p_s = 0.7125\ \mathrm{kip/ft^2}$$

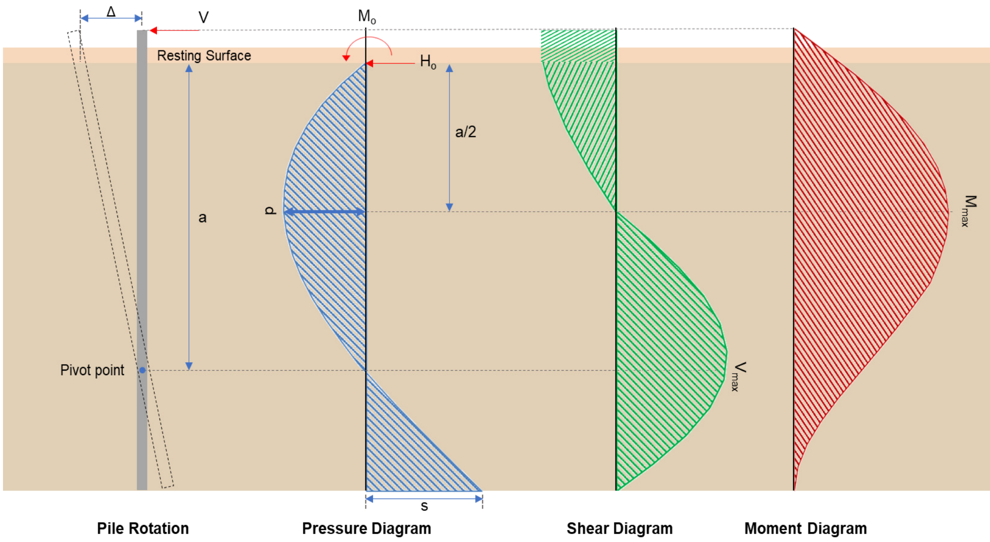
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.60465\ \mathrm{kip/ft^2}\right)}{\left(0.7125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.84864$$

Status: **PASS**
Ratio: **0.850**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.345 \text{ kip})}{1.57 \times (72 \text{ in})}$$

$$H_o = -0.46125 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.553 \text{ kipft}) + ((-4.345 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (72 \text{ in})}$$

$$M_o = 3.0311 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0311 \text{ kipft/ft})}{(-0.46125 \text{ kip/ft})}$$

$$E = 6.5715 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0311 \text{ kipft/ft}) \times (4.75 \text{ ft})) + \left(3 \times (-0.46125 \text{ kip/ft}) \times (4.75 \text{ ft})^2\right)}{(6 \times (3.0311 \text{ kipft/ft})) + (4 \times (-0.46125 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.2954 \text{ ft}$$

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

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

$$V_{max} = ((-0.46125 \text{ kip/ft}) \times (72 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (6.5715 \text{ ft})}{(4.75 \text{ ft})} + 3\right) \times \left(\frac{(3.2954 \text{ ft})}{(4.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (6.5715 \text{ ft})}{(4.75 \text{ ft})} + 2\right) \times \left(\frac{(3.2954 \text{ ft})}{(4.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 8.6 \text{ kip}$$

$V_{max} = 0.0 \text{ kip}$

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

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

$$M_{max} = ((-0.46125 \text{ kip/ft}) \times (72 \text{ in}) \times (4.75 \text{ ft})) \times \left[\left(\frac{(6.5715 \text{ ft})}{(4.75 \text{ ft})} + \frac{(3.2954 \text{ ft})}{2 \times (4.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (6.5715 \text{ ft})}{(4.75 \text{ ft})} + 3\right) \times \left(\frac{(3.2954 \text{ ft})}{2 \times (4.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (6.5715 \text{ ft})}{(4.75 \text{ ft})} + 2\right) \times \left(\frac{(3.2954 \text{ ft})}{2 \times (4.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 19.235 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\, b}$$

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (72 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (72 \text{ in})}$$

$$M_o = 0.00010616 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00010616 \text{ kipft/ft}) \times (4.75 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (4.75 \text{ ft})^2\right)}{(6 \times (0.00010616 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (4.75 \text{ ft}))}$$

$$a = 3.1667 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o\, b}{L_e}\right) \left(\frac{a}{L_e} - 1\right) \left(\frac{a}{L_e}\right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00010616 \text{ kipft/ft}) \times (72 \text{ in})}{(4.75 \text{ ft})}\right) \times \left(\frac{(3.1667 \text{ ft})}{(4.75 \text{ ft})} - 1\right) \times \left(\frac{(3.1667 \text{ ft})}{(4.75 \text{ ft})}\right)^2$$

$$V_{max} = 0.00023839 \text{ kip}$$

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

$$M_{max} = (M_o\, b) \left[1 - \left(4\, \frac{a}{2\, L_e}^3\right) + \left(3\, \frac{a}{2\, L_e}^4\right)\right]$$

$$M_{max} = ((0.00010616 \text{ kipft/ft}) \times (72 \text{ in})) \times \left[1 - \left(4 \times \frac{(3.1667 \text{ ft})}{2 \times (4.75 \text{ ft})}^3\right) + \left(3 \times \frac{(3.1667 \text{ ft})}{2 \times (4.75 \text{ ft})}^4\right)\right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5$ ksi - Concrete strength,
$f_{yk} = 60$ 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,
$A_g = 5184 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.907\ \text{kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5\ \text{ksi}) \times \left(5184\ \text{in}^2\right)\right)}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, \left(0.08 \times \left(5184\ \text{in}^2\right)\right)\right]$$

$$A_{st,required} = -190.08\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-190.08\ \text{in}^2\right), \left(0.0018 \times \left(5184\ \text{in}^2\right)\right)\right]$$

$$A_{min} = 9.3312\ \text{in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(9.3312\ \text{in}^2\right)}{\left(0.3068\ \text{in}^2\right)}$$

$$n_{rebar} = 32$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (32) \times \frac{\pi \times (0.625\ \text{in})^2}{4}$$

$$A_{st} = 9.8175\ \text{in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(9.3312\ \text{in}^2\right)}{\left(9.8175\ \text{in}^2\right)}$$

$$Ratio = 0.95047$$

Status: **PASS**
Ratio: **0.950**

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,

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

$$s_{ties} = Min\left[(16 \times (0.625\ \text{in})), (48 \times (0.375\ \text{in})), Min\,((72\ \text{in}), (72\ \text{in}))\right]$$

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

Summary:

Main reinforcement: **32 - #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

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

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(5184\ \text{in}^2\right) - \left(9.8175\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(9.8175\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 6023.8\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.907 \text{ kip})}{(6023.8 \text{ kip})}$$

$$Ratio = 0.0013126$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 72 \text{ in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (72 \text{ in})$$

$$d = 57.6 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(57.6 \text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.54393$$

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

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

$$V_{c,max} = 5 \times (0.54393) \times \sqrt{(2500 \text{ psi})} \times (72 \text{ in}) \times (57.6 \text{ in})$$

$$V_{c,max} = 563.94 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.907 \text{ kip} \rightarrow 7907 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.54393) \times \sqrt{(2500 \text{ psi})} + \frac{(7907 \text{ lbf})}{6 \times (5184 \text{ in}^2)}\right] \times (72 \text{ in}) \times (57.6 \text{ in})$$

$$V_{c,a} = 226.63 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.54393) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (72 \text{ in}) \times (57.6 \text{ in})$$

$$V_{c,b} = 743.98 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(563.94 \text{ kip}), (226.63 \text{ kip}), (743.98 \text{ kip})\right]$$

$$V_c = 226.63 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (72 \text{ in}) \times (57.6 \text{ in})$$

	$$V_{s,a} = 1658.9 \text{ kip}$$ 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) $$V_{s,b} = \frac{2 \, A_v \, f_{ywk} \, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (57.6 \text{ in})}{(10 \text{ in})}$$ $$V_{s,b} = 76.341 \text{ kip}$$ V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(1658.9 \text{ kip}), (76.341 \text{ kip})\right]$$ $$V_s = 76.341 \text{ kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi \, (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((226.63 \text{ kip}) + (76.341 \text{ kip}))$$ $$\phi V_n = 196.93 \text{ kip}$$ **Considering x-direction:** $V_{max} = 8.6 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(8.6 \text{ kip})}{(196.93 \text{ kip})}$$ $$Ratio = 0.043669$$	Status: **PASS** Ratio: **0.040**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b \, D^2}{6}$$ $$S_m = \frac{(72 \text{ in}) \times (72 \text{ in})^2}{6}$$ $$S_m = 62208 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 62208.000in^3$$ $$\phi M_{n,1} = 842.400kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi \, 0.85 \, f'_{ck} \, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(62208 \text{ in}^3\right)$$ $$\phi M_{n,2} = 7160.4 \text{ kipft}$$	

	Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(842.4 \text{ kipft}), (7160.4 \text{ kipft})]$$ $$\phi M_n = 842.4 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 19.235 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(19.235 \text{ kipft})}{(842.4 \text{ kipft})}$$ $$Ratio = 0.022833$$	Status: **PASS** Ratio: **0.020**
	Considering z-direction: $M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.00056617 \text{ kipft})}{(842.4 \text{ kipft})}$$ $$Ratio = 672.09 \times 10^{-9}$$	Status: **PASS** Ratio: **0.000**