Project Details

Project Name: Tiskus - Easthampton DE RE 4x4 Struts 15deg 6ft - V1Jb-

Location: 94 West St, Easthampton, MA 01027, USA

Unique ID: 1P-0-8TOP-XD-84-L-4Hx4W-STRUTS-90B3

Dealer: ______________________________

Date: Thu May 15 2025

Number of Modules: 16

Number of Poles: 1

Date Sold:

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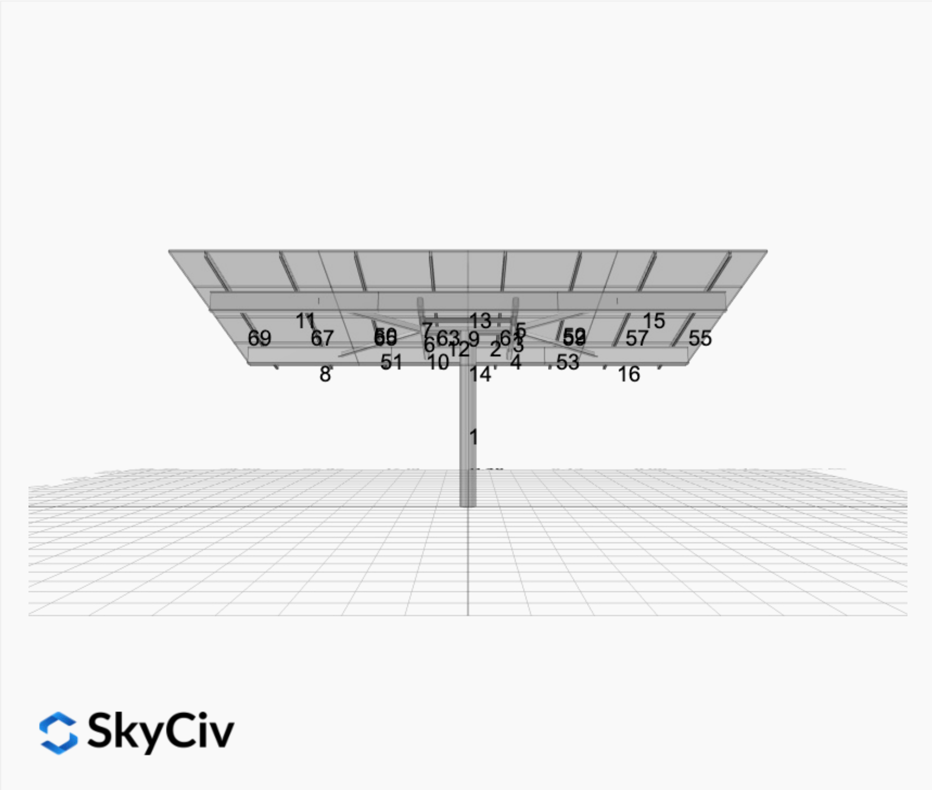


Array Dimensions N/S	15.04 ft
Array Dimensions E/W	22.92 ft
Winter Tilt Angle	15
Front Edge Clearance	6 ft

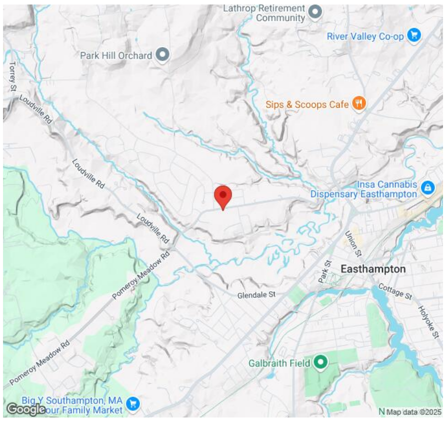
MT Solar Bill of Materials (1P-0-8TOP-XD-84-L-4Hx4W-STRUTS-90B3)

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	4

Rail Bill of Materials

Part	Qty
Rails (180in)	8
Rail Attachment	16
Module Mid Clamp	24
Module End Clamp	16
Ground Lug	4

Site Details:



Site Address: 94 West St, Easthampton, MA 01027, USA

Array Specification

Duty Classification:	XD
Module Width:	44.61 in
Module Length:	67.76in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Winter Tilt Angle:	15
Front Edge Clearance:	6
Total Array Height at Tilt:	9.89 ft
Total Frame Length:	21.50 ft
Module Info/Notes:	SILFAB SIL-440 QD
Array Dimensions N/S:	15.04 ft
Array Dimensions E/W:	22.92 ft
Rail Length:	180.44 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	7.95 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.00 ft
Foundation Volume:	2.094 y^3

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	94 West St, Easthampton, MA 01027, USA
Wind Speed:	114 mph
Snow Load:	40 psf

Design Disclaimer

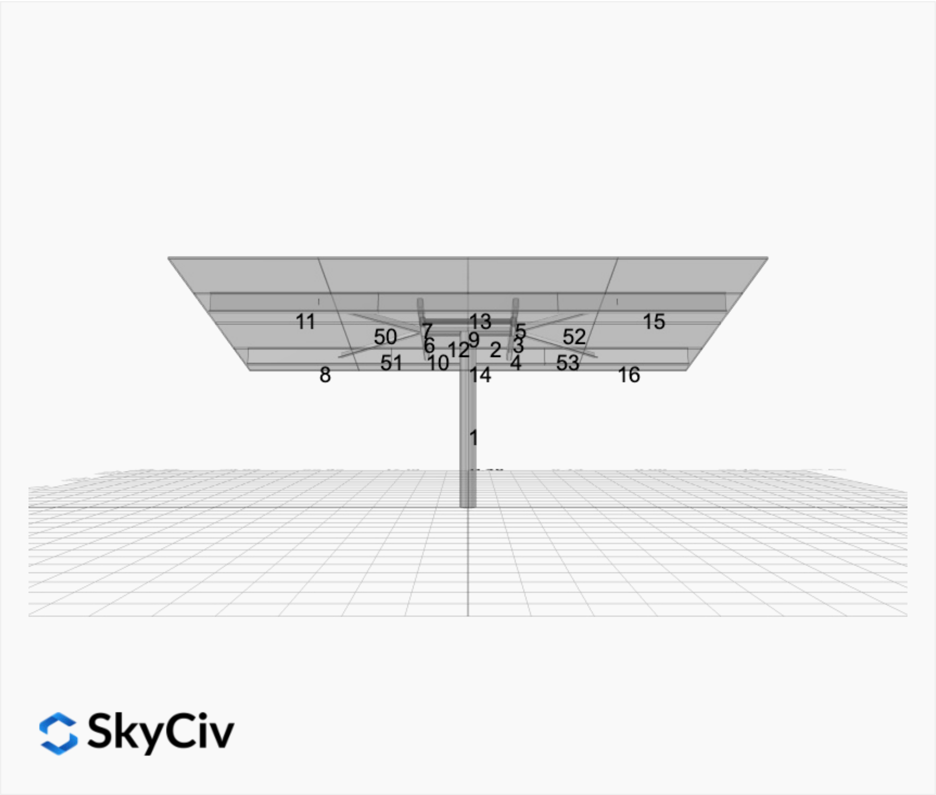
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

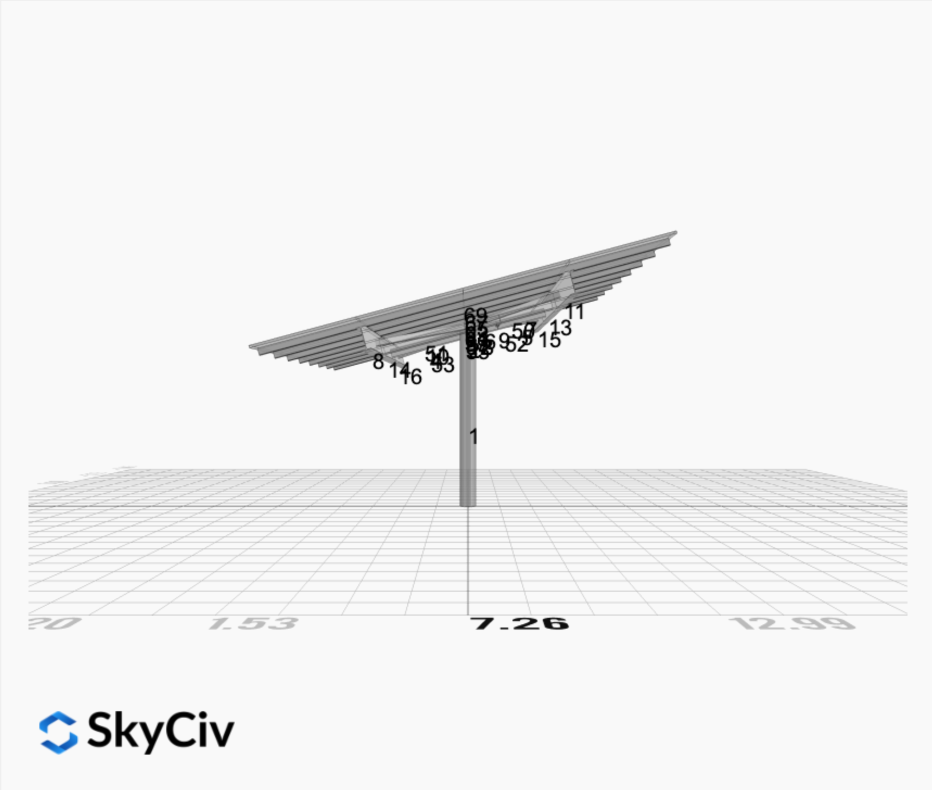
AutoDesigner Input

{"wind_speed_override":114,"snow_load_override":40,"direct_snow_load":false,"add_angle_brace":true,"product_type":"Beam","designer_name":"Robin Creamer","designer_email":"robinc@pvsquared.coop","designer_phone":"4135224862","project_id":"Tiskus - Easthampton DE RE 4x4 Struts 15deg 6ft - V1Jb-","site_address":"94 West St, Easthampton, MA 01027, USA","module_info":"SILFAB SIL-440 QD","module_width":44.61,"module_length":67.76,"number_rows":"4","number_columns":4,"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":15,"ground_clearance":6,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"8_40","soil_type":"sand","customer_foundation_override":"36_Round","foundation_type":"Round","foundation_size":36,"check_rails":false}

Design Notes:

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





1
8
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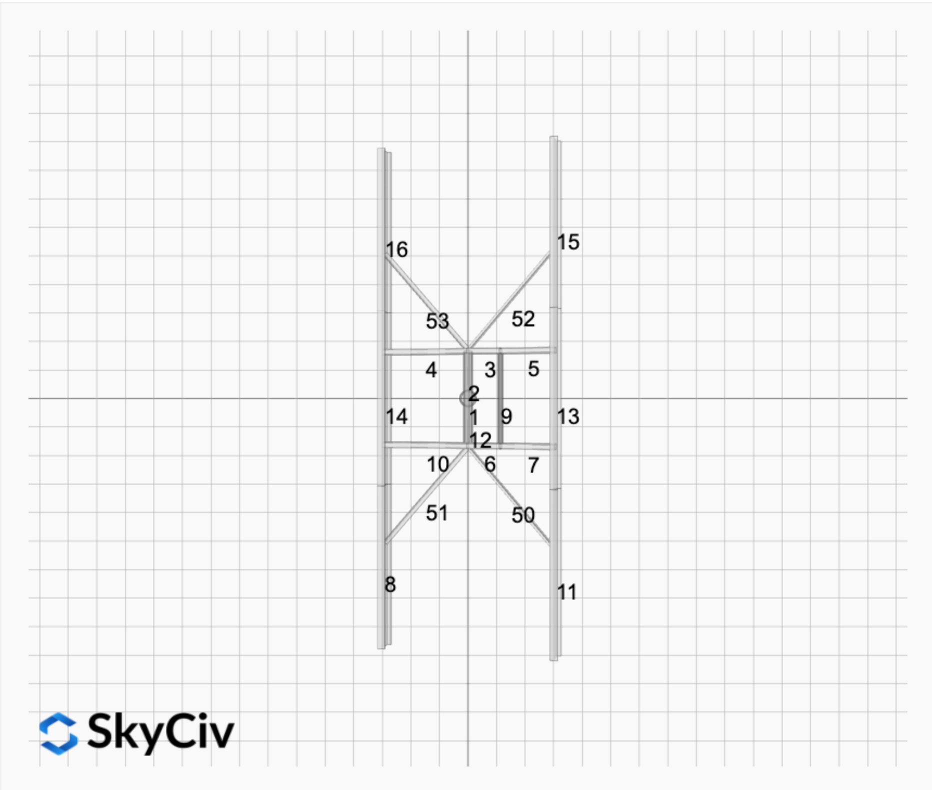
11
13
15
52
53
1.53
7.26
12.99
SkyCiv



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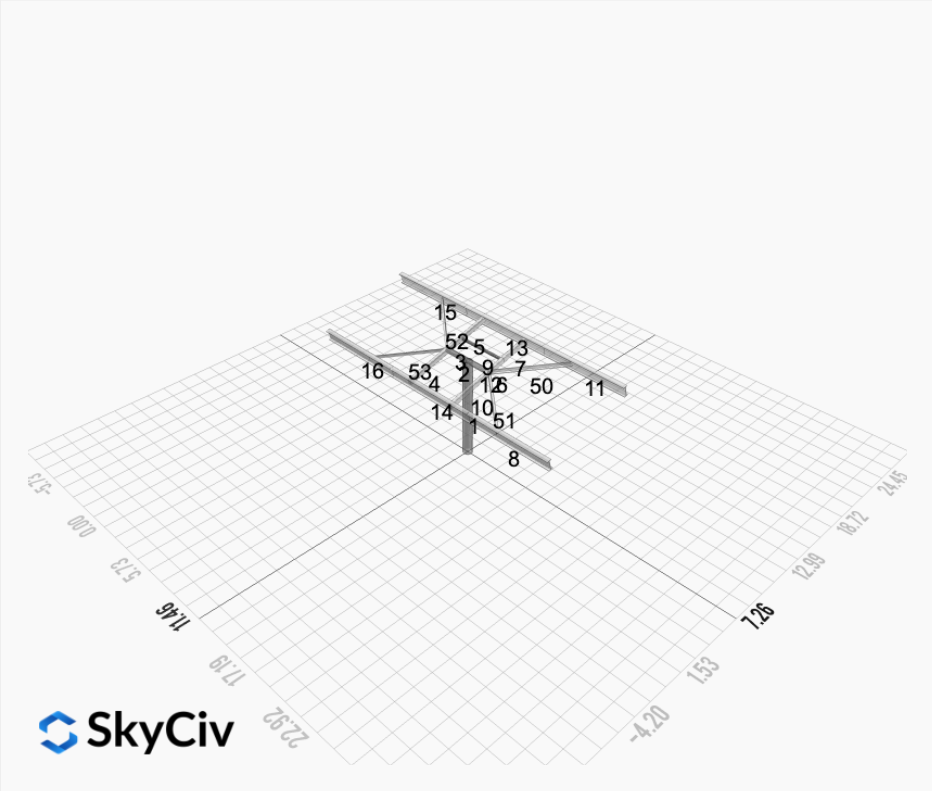
SkyCiv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11
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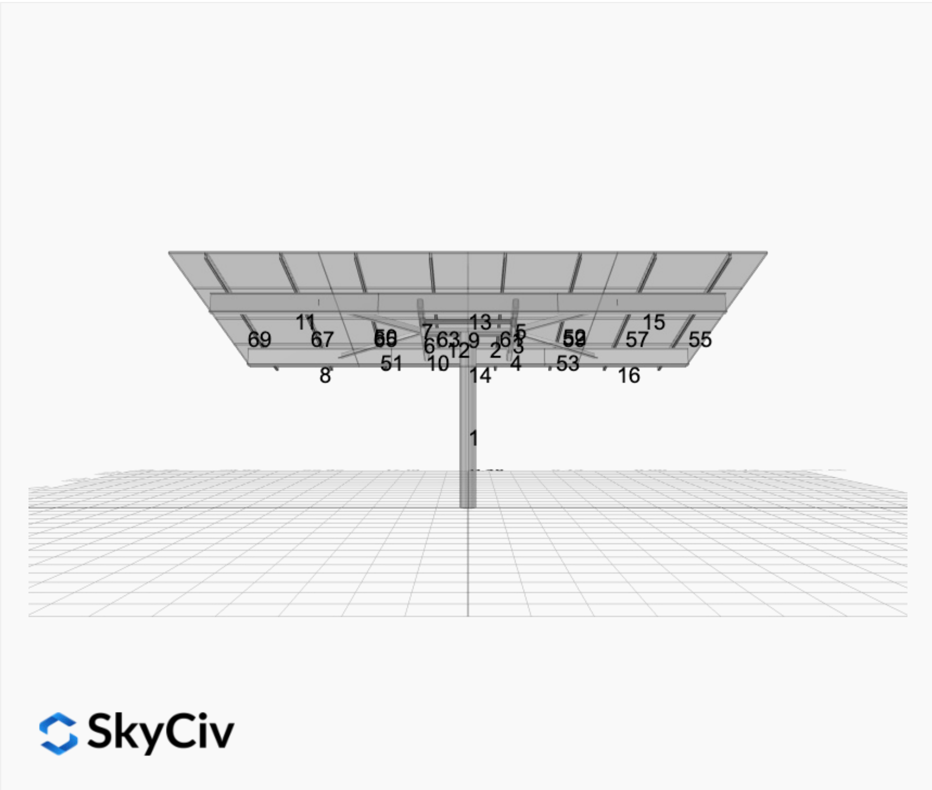
5.73
11.46
17.19
22.92
24.45
18.72
12.99
7.26
1.53
-4.20
SkyCiv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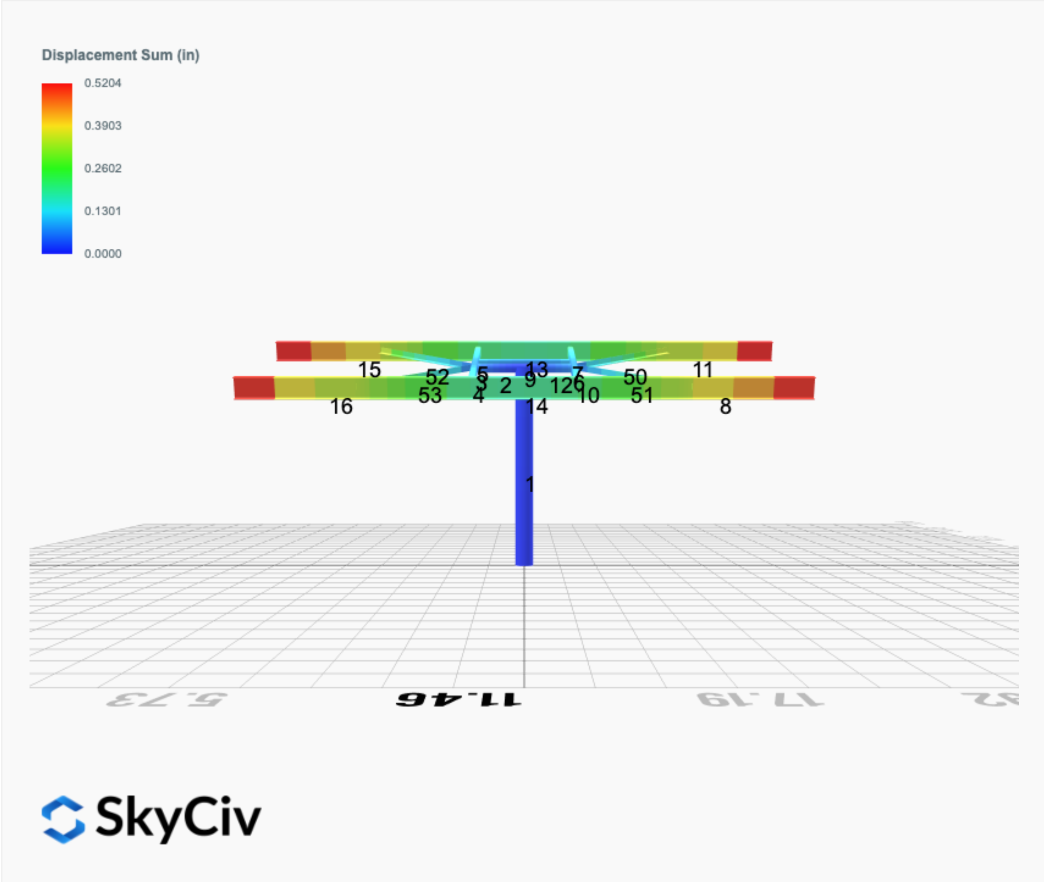
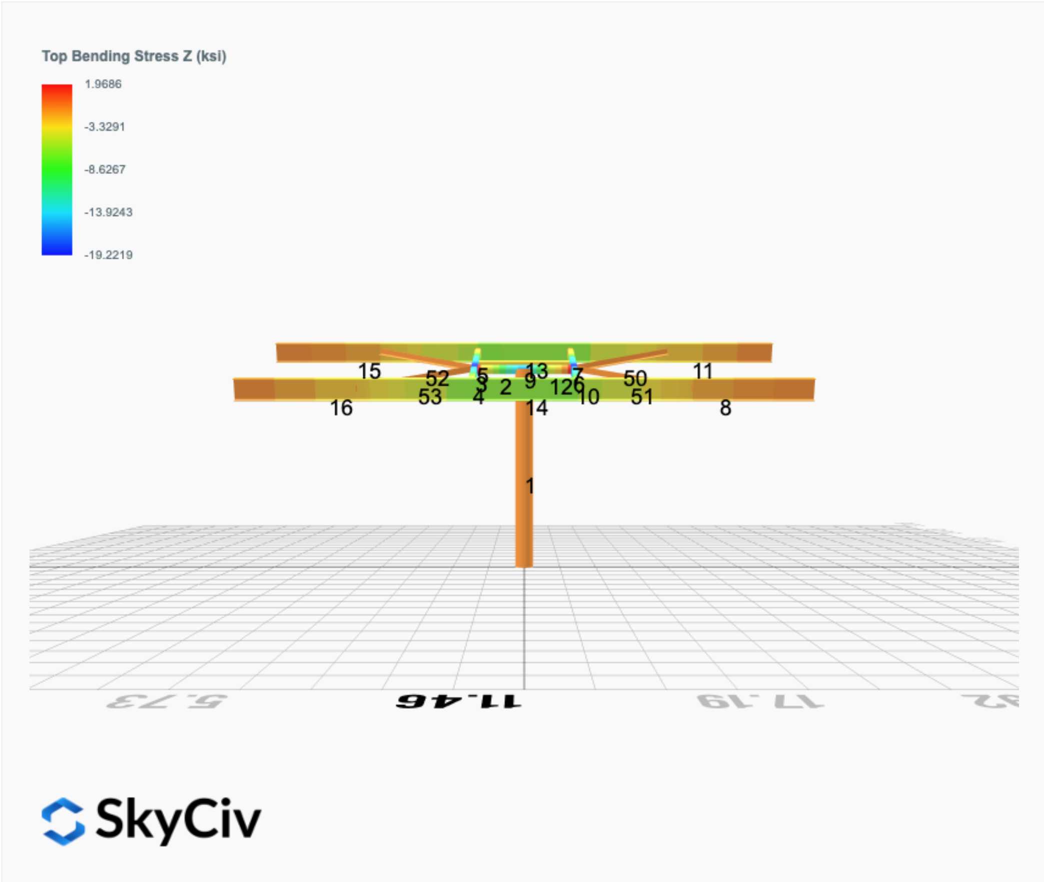
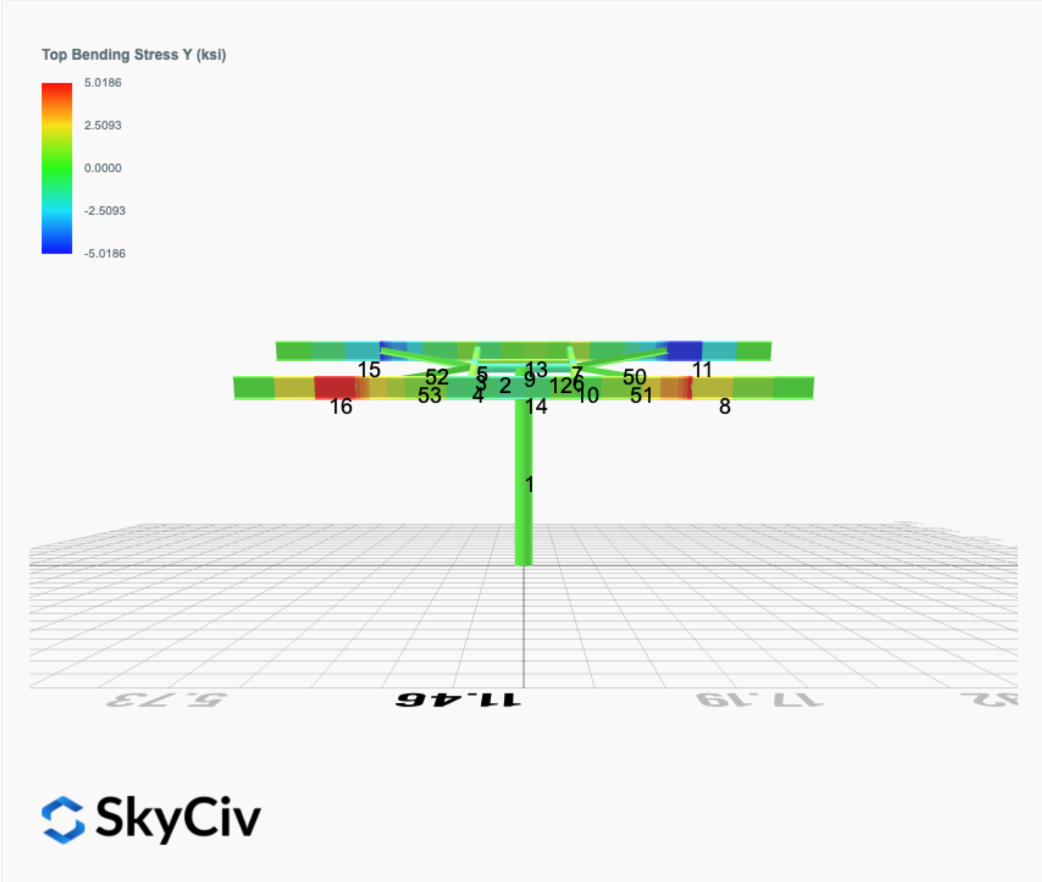
8
16
1
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
5.0186
2.5093
0.0000
-2.5093
-5.0186
15
52
5
13
9
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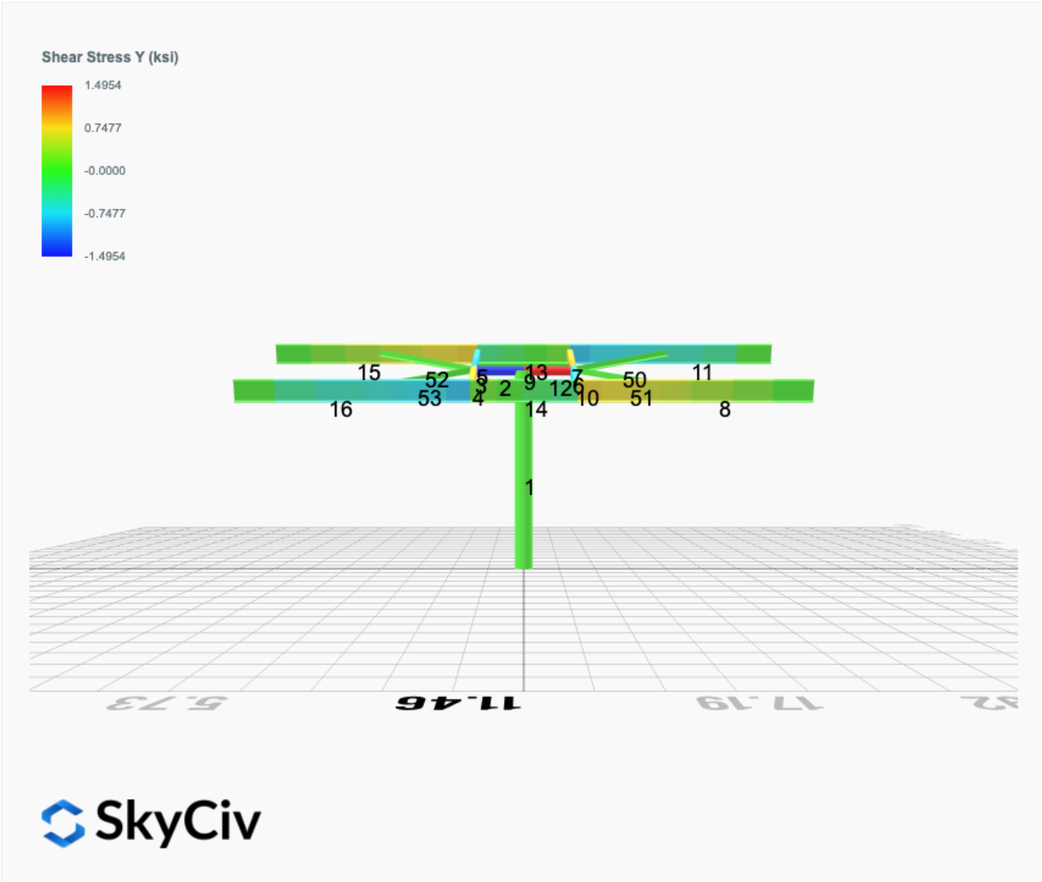
51
8
14
1
11.46
SkyCiv


Shear Stress Y (ksi)
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0.7477
-0.0000
-0.7477
-1.4954
15
52
5
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12
6
50
11
16
53
4
2
10
51
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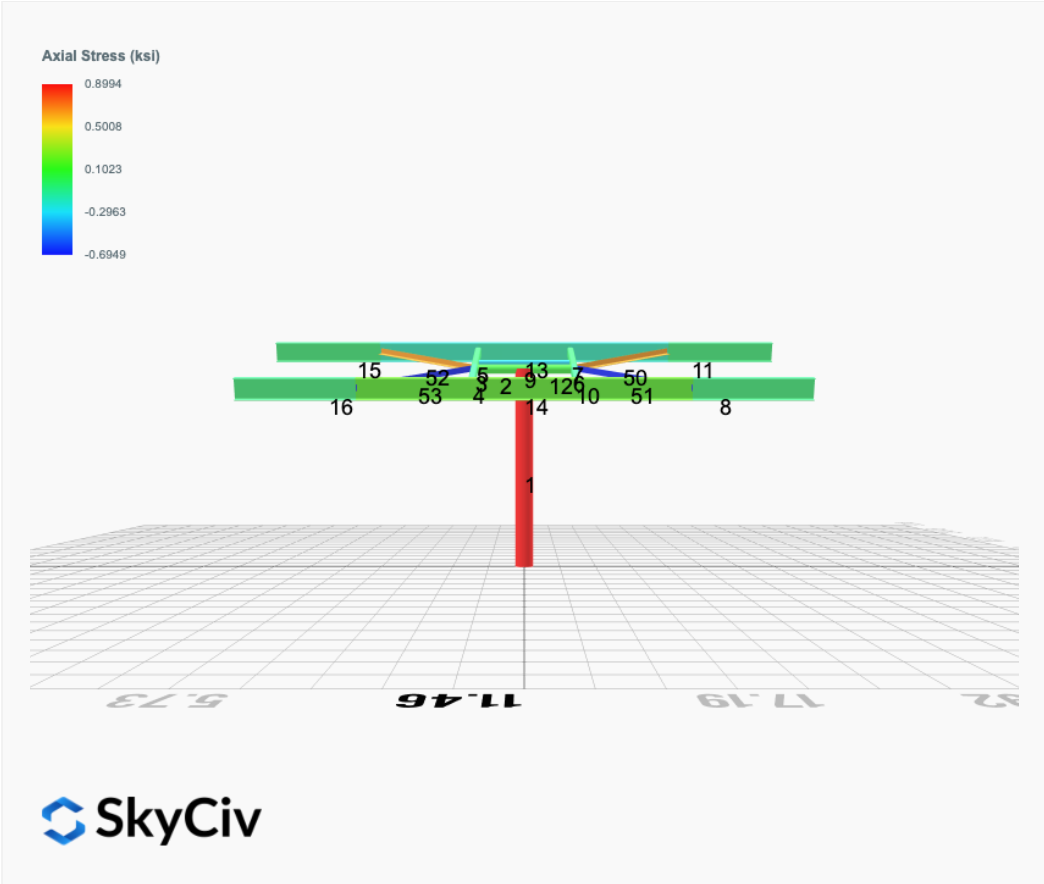
14
1
11.46
SkyCiv


Axial Stress (ksi)
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0.5008
0.1023
-0.2963
-0.6949
15
52
5
13
7
50
11
3
2
9
12
6
53
4
14
10
51
16
8
1
11.46
5.73
17.19
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	2.7050	0.0000	-0.0000	-0.0000	0.0335
ULS: 2. D + L	0.0000	2.7050	0.0000	-0.0000	-0.0000	0.0335
ULS: 3. D + (S or Lr or R)	0.0000	10.2596	0.0000	-0.0000	-0.0000	0.0793
ULS: 3. D + (S or Lr or R)	0.0000	2.7050	0.0000	-0.0000	-0.0000	0.0335
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	8.3710	0.0000	-0.0000	-0.0000	0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7050	0.0000	-0.0000	-0.0000	0.0335
ULS: 5b. D + 0.7E	0.0000	2.7050	0.0000	-0.0000	-0.0000	0.0335
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	8.3710	0.0000	-0.0000	-0.0000	0.0679
ULS: 8. 0.6D + 0.7E	0.0000	1.6230	0.0000	-0.0000	-0.0000	0.0201
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0651	6.6802	0.0000	-0.0000	-0.0000	10.1993
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0651	6.6802	0.0000	-0.0000	-0.0000	10.1993
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8080	-0.3106	0.0000	-0.0000	-0.0000	-4.2342
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6979	0.1007	0.0000	-0.0000	-0.0000	-15.4991
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7989	11.3523	0.0000	-0.0000	-0.0000	7.6922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7989	11.3523	0.0000	-0.0000	-0.0000	7.6922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6060	6.1093	0.0000	-0.0000	-0.0000	-3.1329
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5234	6.4177	0.0000	-0.0000	-0.0000	-11.5816
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7989	5.6864	0.0000	-0.0000	-0.0000	7.6578
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7989	5.6864	0.0000	-0.0000	-0.0000	7.6578
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6060	0.4433	0.0000	-0.0000	-0.0000	-3.1672
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5234	0.7518	0.0000	-0.0000	-0.0000	-11.6159
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0651	5.5982	0.0000	-0.0000	-0.0000	10.1859
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0651	5.5982	0.0000	-0.0000	-0.0000	10.1859
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8080	-1.3926	0.0000	-0.0000	-0.0000	-4.2476
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6979	-0.9814	0.0000	-0.0000	-0.0000	-15.5125

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	18.6460
Shear X	-1.7752
Shear Z	0.0000
Moment X	-0.0002
Moment Y (Twist)	0.0005
Moment Z	26.1901

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	11.3523
Shear X	-1.0651
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	15.5125

Project Details

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

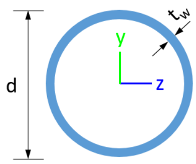
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

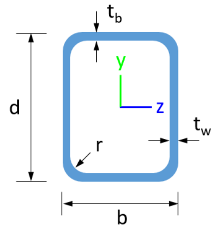
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

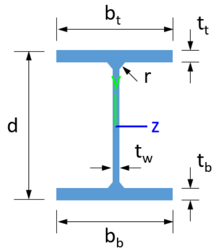
Section Dimensions



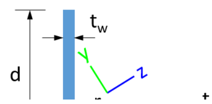
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					

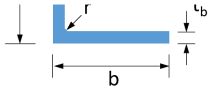


ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



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





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)
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
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	9	16.69	16.69	7.95	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.18	300	200	1
4	17	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,2.19,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.57,1.68,1.67,1.67,1.66,1.68	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.73,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.23,1.17,1.18,1.18,1.17,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.73,1.65,1.67,1.67,1.66,1.66	300	200	1
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9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,2.19,1.67,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.57,1.68,1.67,1.67,1.66,1.68	300	200	1
11	20	7.00	7.00	7.00	2.31,2.30,2.31,2.30,2.31,2.31,2.30,2.30,2.32,2.29,2.30,2.30,2.28,2.29,2.30,2.30,2.30,2.30,2.30,2.30,2.40,2.28,2.30,2.30,2.29,2.29	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.11,1.02,1.02,1.02,1.04,1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.08,1.02,1.02,1.02,1.03	300	200	1
14	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.00,1.03,1.02,1.02,1.02,1.11,1.02,1.02,1.02,1.02,1.02,1.02,1.08,1.04,1.02,1.02,1.02,1.16	300	200	1
15	20	7.00	7.00	7.00	2.31,2.30,2.31,2.30,2.31,2.31,2.30,2.30,2.32,2.29,2.30,2.30,2.28,2.29,2.30,2.30,2.30,2.30,2.30,2.30,2.40,2.28,2.30,2.30,2.29,2.29	300	200	1
16	20	7.00	7.00	7.00	2.31,2.30,2.31,2.30,2.31,2.31,2.30,2.30,3.20,2.30,2.30,2.30,2.29,2.31,2.30,2.30,2.30,2.30,2.30,2.30,2.23,2.31,2.30,2.30,2.29,2.31	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	377.97	269.11	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	31.12	6.46	56.26	44.91
14	159.30	97.43	30.69	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
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	0.069	0.314	0.000	0.016	0.000	0.319	#16	0.341	Not Required	Pass
2	0.004	0.601	0.075	0.122	0.012	0.667	#21	0.054	Not Required	Pass
3	0.001	0.845	0.074	0.085	0.032	0.919	#21	0.046	Not Required	Pass
4	0.001	0.805	0.022	0.081	0.002	0.828	#21	0.082	Not Required	Pass
5	0.001	0.524	0.019	0.084	0.007	0.541	#21	0.076	Not Required	Pass
6	0.001	0.845	0.074	0.085	0.032	0.920	#21	0.046	Not Required	Pass
7	0.001	0.524	0.019	0.084	0.007	0.541	#21	0.076	Not Required	Pass
8	0.014	0.210	0.138	0.050	0.010	0.235	#21	0.535	Not Required	Pass
9	0.015	0.102	0.029	0.001	0.000	0.139	#21	0.137	Not Required	Pass
10	0.001	0.806	0.022	0.081	0.002	0.828	#21	0.082	Not Required	Pass
11	0.006	0.219	0.138	0.052	0.010	0.233	#21	0.357	Not Required	Pass
12	0.004	0.601	0.075	0.122	0.012	0.667	#21	0.054	Not Required	Pass
13	0.006	0.533	0.026	0.065	0.005	0.550	#21	0.204	Not Required	Pass
14	0.010	0.517	0.023	0.062	0.004	0.528	#21	0.306	Not Required	Pass
15	0.006	0.219	0.138	0.052	0.010	0.233	#21	0.357	Not Required	Pass
16	0.014	0.210	0.138	0.050	0.010	0.235	#21	0.357	Not Required	Pass
50	0.150	0.010	0.005	0.002	0.001	0.163	#6	0.783	Not Required	Pass
51	0.030	0.009	0.013	0.002	0.001	0.047	#23	0.522	Not Required	Pass
52	0.150	0.010	0.005	0.002	0.001	0.163	#24	0.783	Not Required	Pass
53	0.030	0.009	0.013	0.002	0.001	0.047	#24	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

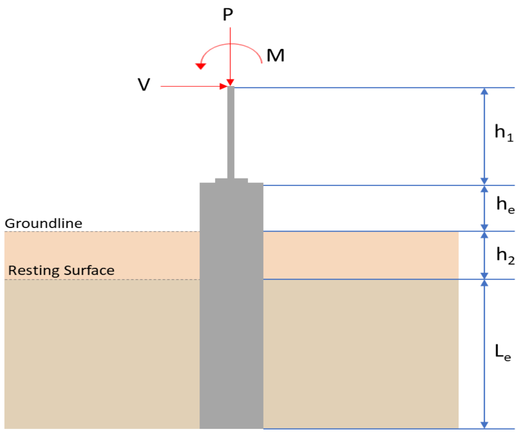
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 8\ \mathrm{ft}$ - 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)	11.352	18.646
V_x (kip)	-1.065	-1.775
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	15.512	26.190

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,

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

$$H_o = \frac{(-1.065\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.355\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.512 \text{ kipft}) + ((-1.065 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1707 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.3844 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(7.3844 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.384 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.384 \text{ ft})}{(8 \text{ ft})}$$

$$Ratio = 0.923$$

Status: **PASS**
Ratio: **0.920**

End-bearing Capacity (ASD)

A - Pile cros-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.352 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.606 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.606 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.80299$$

Status: **PASS**
Ratio: **0.800**

Czerniak **Lateral Soil Pressure (ASD):**

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.6667$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.355 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 5.1707 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1707 \text{ kipft/ft}) \times (8 \text{ ft})) + \left(3 \times (-0.355 \text{ kip/ft}) \times (8 \text{ ft})^2\right)}{(6 \times (5.1707 \text{ kipft/ft})) + (4 \times (-0.355 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.512 \text{ ft}$$

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

$$p = \frac{1.178\, [(4\, M_o) + (3\, H_o\, L_e)]^2}{L_e^2\, [(3\, M_o) + (2\, H_o\, L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.1707 \text{ kipft/ft})) + (3 \times (-0.355 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (5.1707 \text{ kipft/ft})) + (2 \times (-0.355 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.27694 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

$$s = \frac{9.425\, [(2\, M_o) + (H_o\, L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.1707 \text{ kipft/ft})) + ((-0.355 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1047 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.512 \text{ ft})}{2}$$

$$p_a = 0.4134 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.27694 \text{ kip/ft}^2\right)}{\left(0.4134 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.6699$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

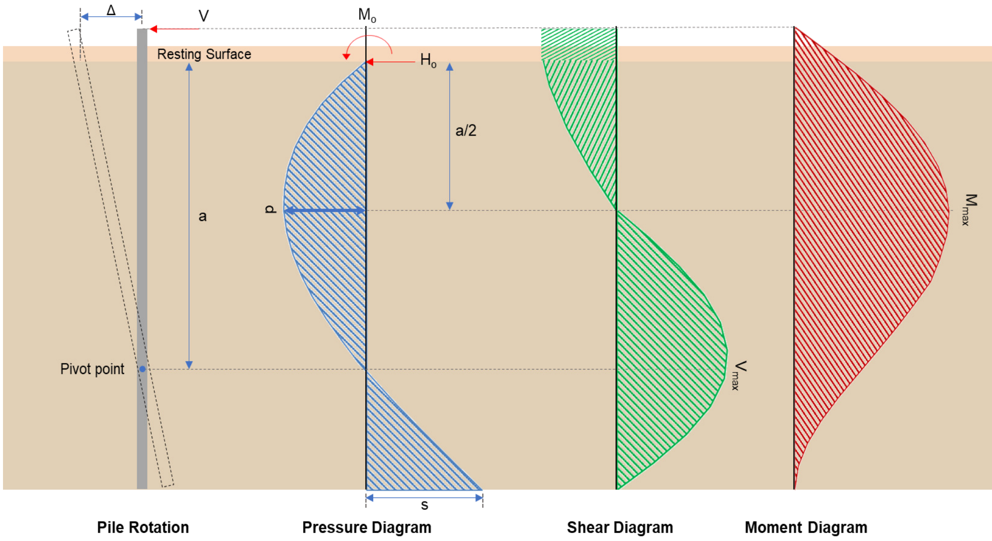
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.1047 \text{ kip/ft}^2\right)}{}$$

$$Ratio = \frac{\cdots}{(1.2\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.92057$$

Status: PASS
Ratio: 0.920



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.775\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.59167\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.19\ \mathrm{kipft}) + ((-1.775\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(36\ \mathrm{in})}$$

$$M_o = 8.73\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.73\ \mathrm{kipft/ft})}{(-0.59167\ \mathrm{kip/ft})}$$

$$E = 14.755\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.73\ \mathrm{kipft/ft}) \times (8\ \mathrm{ft})) + \left(3 \times (-0.59167\ \mathrm{kip/ft}) \times (8\ \mathrm{ft})^2\right)}{(6 \times (8.73\ \mathrm{kipft/ft})) + (4 \times (-0.59167\ \mathrm{kip/ft}) \times (8\ \mathrm{ft}))}$$

$$a = 5.5103\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.59167\ \mathrm{kip/ft}) \times (36\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.755\ \mathrm{ft})}{(8\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.5103\ \mathrm{ft})}{(8\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (14.755\ \mathrm{ft})}{(8\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.5103\ \mathrm{ft})}{(8\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.9641 \text{ kip}$$

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

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

$$M_{max} = ((-0.59167 \text{ kip/ft}) \times (36 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(14.755 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5103 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (14.755 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5103 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.755 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5103 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 26.566 \text{ kipft}$$

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 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(18.646 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.79 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max \left[(-36.79 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$Ratio = 0.99533$$

Status: **PASS**
Ratio: **1.000**

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 center-to-center spacing of ties,

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

$$s_{ties} = Min\left[(16 \times (0.625\ \text{in})), (48 \times (0.375\ \text{in})), (36\ \text{in})\right]$$

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

Summary:

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(1017.9\ \text{in}^2\right) - \left(1.8408\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(1.8408\ \text{in}^2\right)\right)\right]$$

$$\phi P_N = 1253.9\ \text{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.646\ \text{kip})}{(1253.9\ \text{kip})}$$

$$Ratio = 0.01487$$

Status: **PASS**
Ratio: **0.010**

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 36\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

$$d = 0.80 \times (36\ \text{in})$$

$$d = 28.8\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\ \text{in})}{10}}}, 1\right]$$

$$\lambda_s = 0.71796$$

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500\ \text{psi})} \times (36\ \text{in}) \times (28.8\ \text{in})$$

$$V_{c,max} = 186.09\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}, P = 18.646\ \text{kip} \rightarrow 18646\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(18646\ \mathrm{lbf})}{6 \times (1017.9\ \mathrm{in}^2)}\right] \times (36\ \mathrm{in}) \times (28.8\ \mathrm{in})$$

$$V_{c,a} = 77.603\ \mathrm{kip}$$

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

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

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500\ \mathrm{psi})} + (0.05 \times (2500\ \mathrm{psi}))\right] \times (36\ \mathrm{in}) \times (28.8\ \mathrm{in})$$

$$V_{c,b} = 204.04\ \mathrm{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(186.09\ \mathrm{kip}), (77.603\ \mathrm{kip}), (204.04\ \mathrm{kip})\right]$$

$$V_c = 77.603\ \mathrm{kip}$$

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

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

$$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (36\ \mathrm{in}) \times (28.8\ \mathrm{in})$$

$$V_{s,a} = 414.72\ \mathrm{kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\,d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$

$$A_v = 0.11045\ \mathrm{in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

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

$$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (28.8\ \mathrm{in})}{(10\ \mathrm{in})}$$

$$V_{s,b} = 38.17\ \mathrm{kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(414.72\ \mathrm{kip}), (38.17\ \mathrm{kip})\right]$$

$$V_s = 38.17\ \mathrm{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.603\ \mathrm{kip}) + (38.17\ \mathrm{kip}))$$

$$\phi V_n = 75.253\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 6.9641\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.9641\ \mathrm{kip})}{(75.253\ \mathrm{kip})}$$

$$Ratio = 0.092542$$

Status: **PASS**
Ratio:

		Ratio: **0.090**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{\pi\, D^3}{32}$$ $$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$ $$S_m = 4580.4 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 4580.442in^3$$ $$\phi M_{n,1} = 62.027kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(4580.4 \text{ in}^3\right)$$ $$\phi M_{n,2} = 527.23 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$ $$\phi M_n = 62.027 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 26.566 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(26.566 \text{ kipft})}{(62.027 \text{ kipft})}$$ $$Ratio = 0.4283$$	Status: **PASS** Ratio: **0.430**