Your Project Calculations

Project Name: ArapahoValleyRanch-JB-RevA-Struts

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=ArapahoValleyRanch-JB-RevA-Struts&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=AdbDoNcFCVOJw3rXVH3vfENBjNCQD8ceGrjOHI8rFUg7yBdv4mhrvWkvFs1wDnlu

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-SD-84-L-5Hx3W-0G8I
Duty Classification:	SD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Desired Tilt Angle:	55
Front Edge Clearance:	5
Total Array Height at Tilt:	19.11 ft
Total Frame Length:	21.50 ft
Frame Weight:	957 lbs
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	22.05 ft
Rail Length:	208.00 in
Rail Spacing:	3.68 ft
Rail Check:	PASS (75% utilized)

Support Specifications

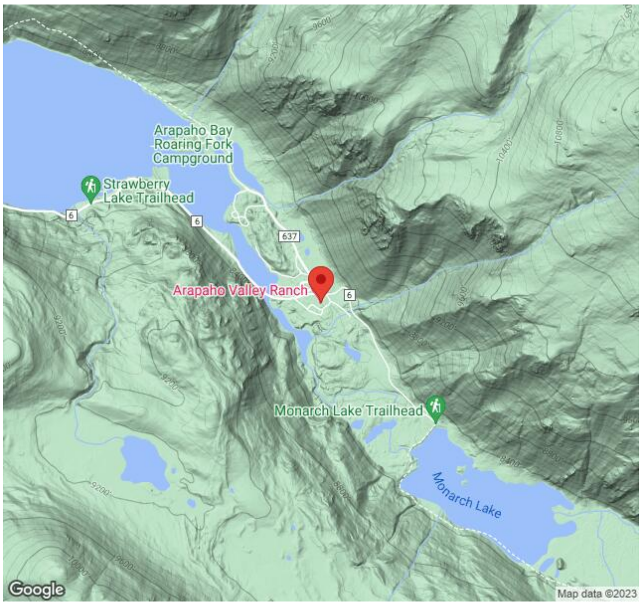
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	12.10 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.00 ft
Foundation Volume:	4.148 y^3
Foundation Result:	PASSED
Mount Twist:	0.000736 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	8992 Co Rd 6, Granby, CO 80446, USA
Wind Speed:	115 mph
Snow Load:	68 psf
Design Uplift Pressure:	0.014991 ksf
Design Downforce Pressure:	-0.014991 ksf
Design Snow Pressure:	0.011216 ksf



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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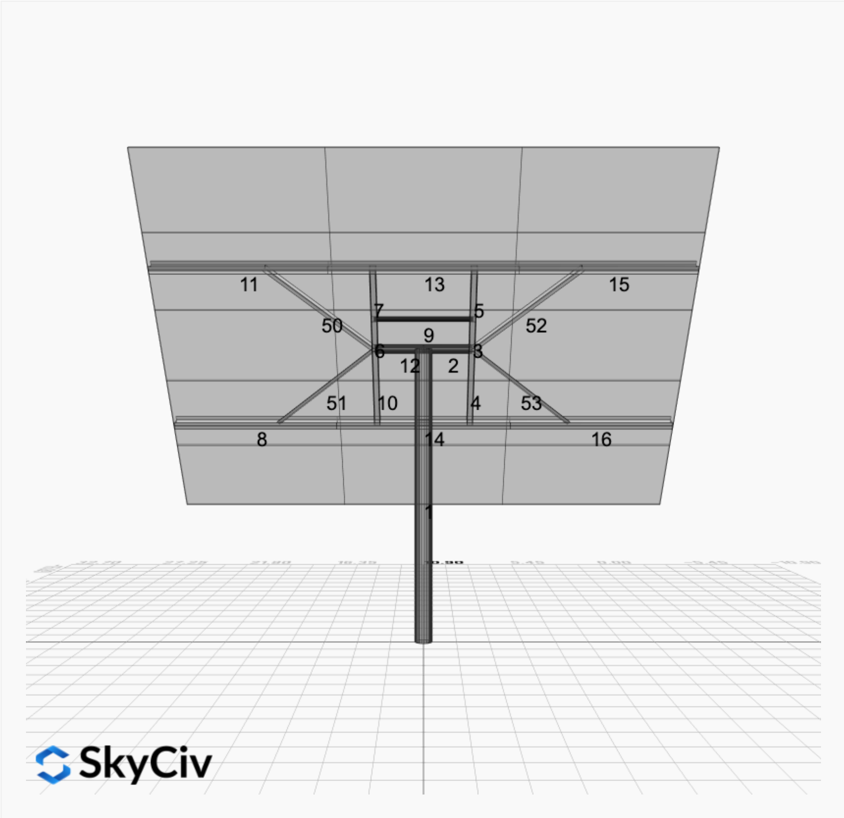
Design Notes:

- 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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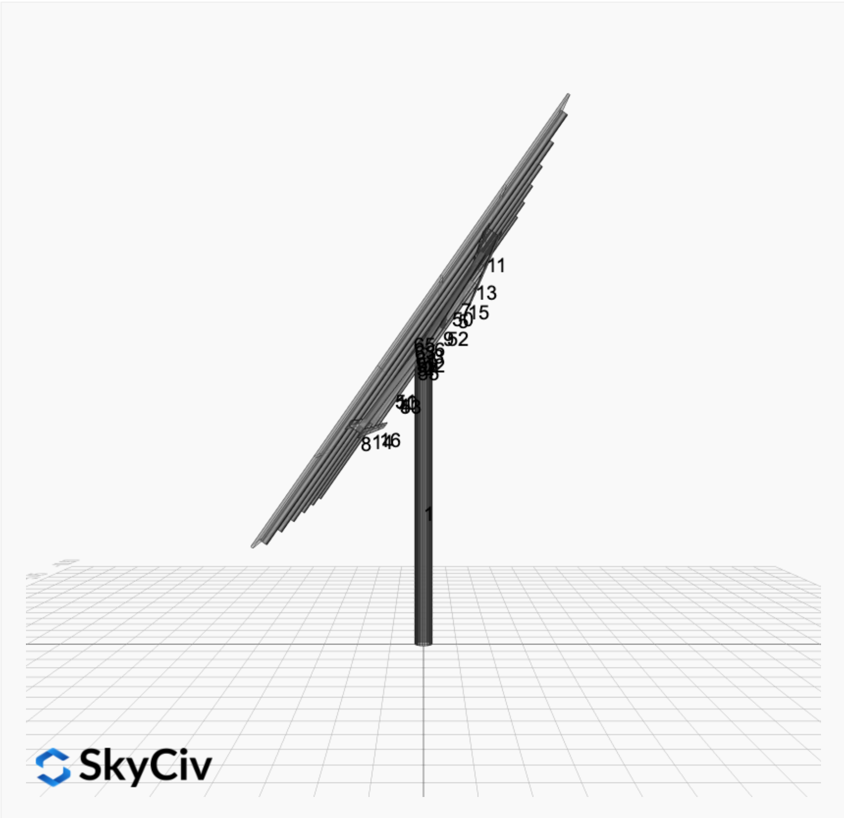
SkyCiv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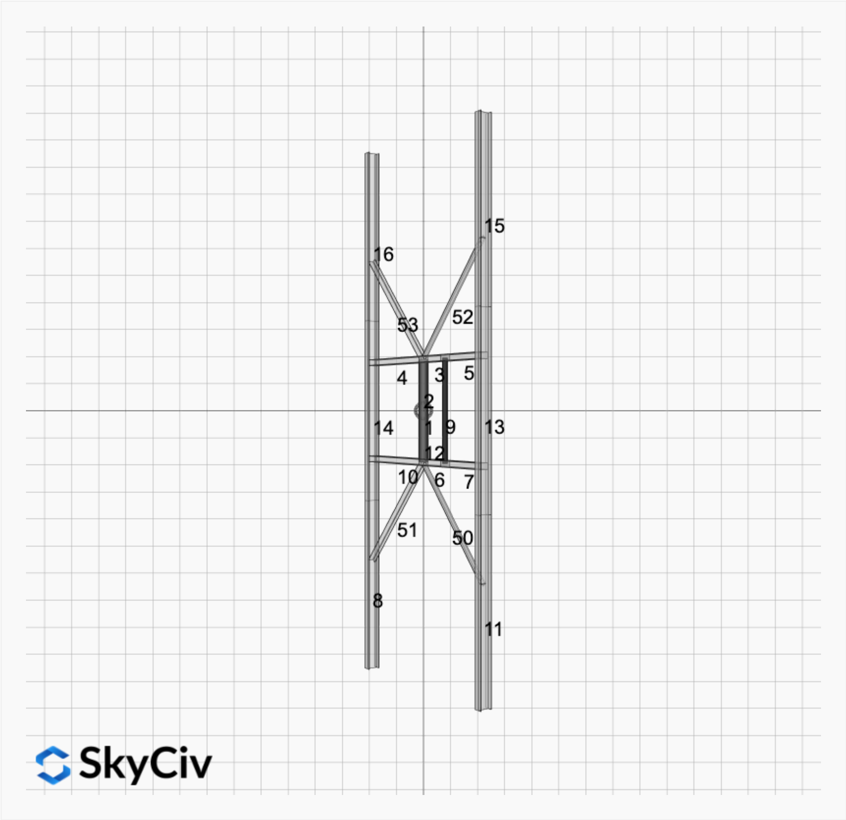
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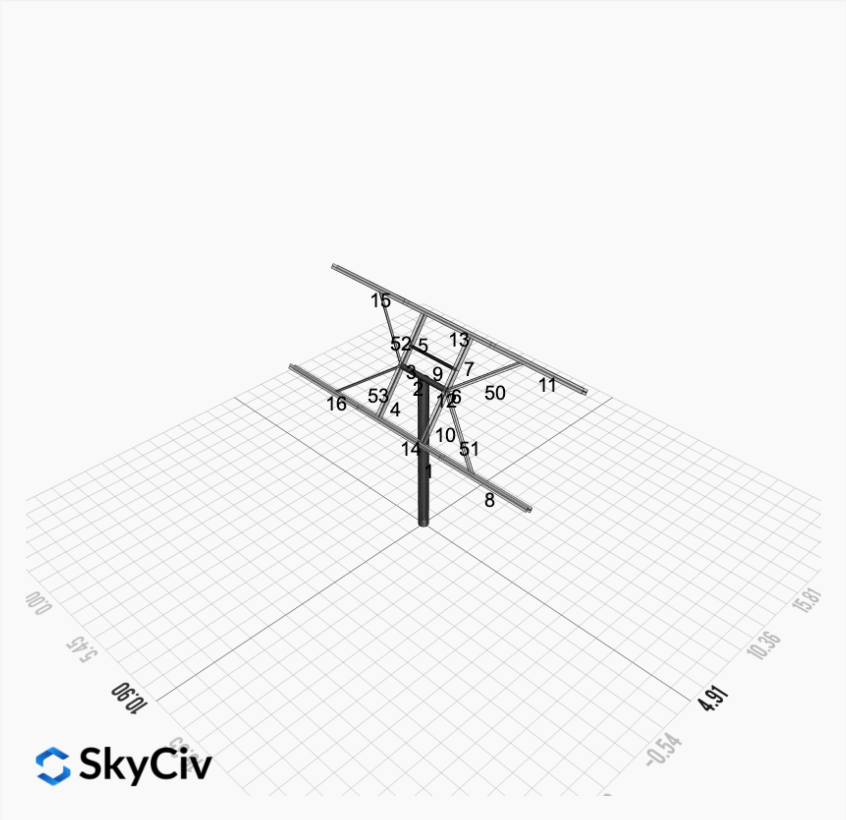
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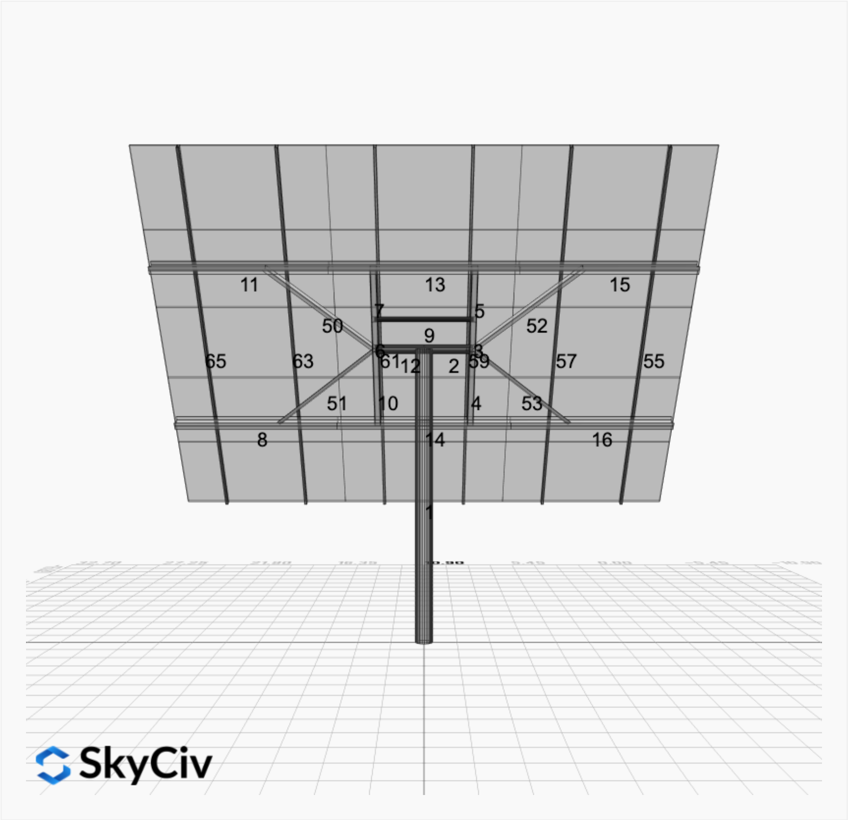

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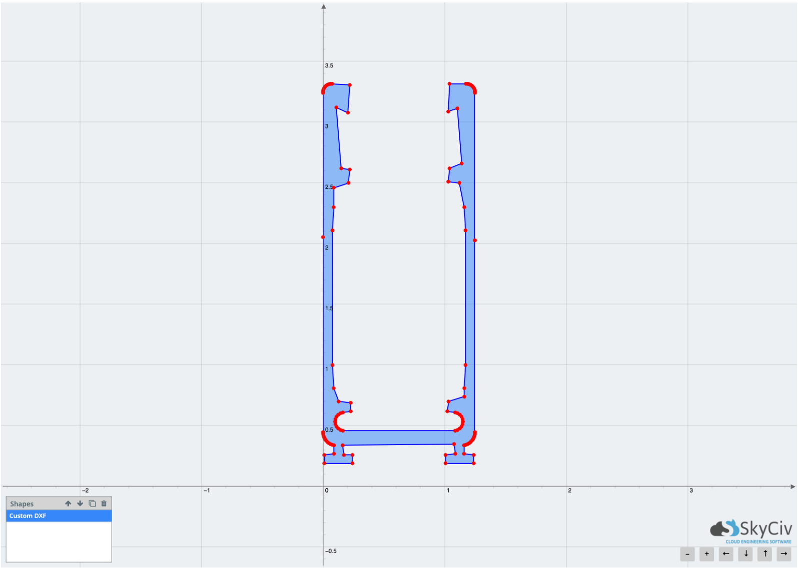
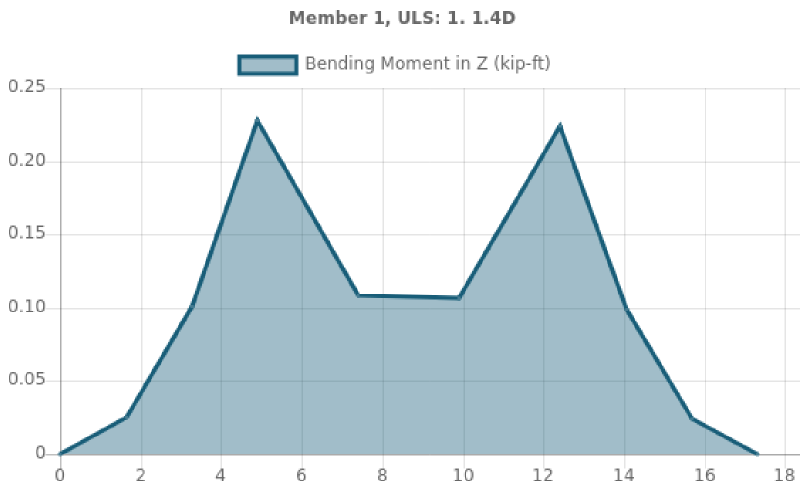
SkyCiv

Rail Design Check

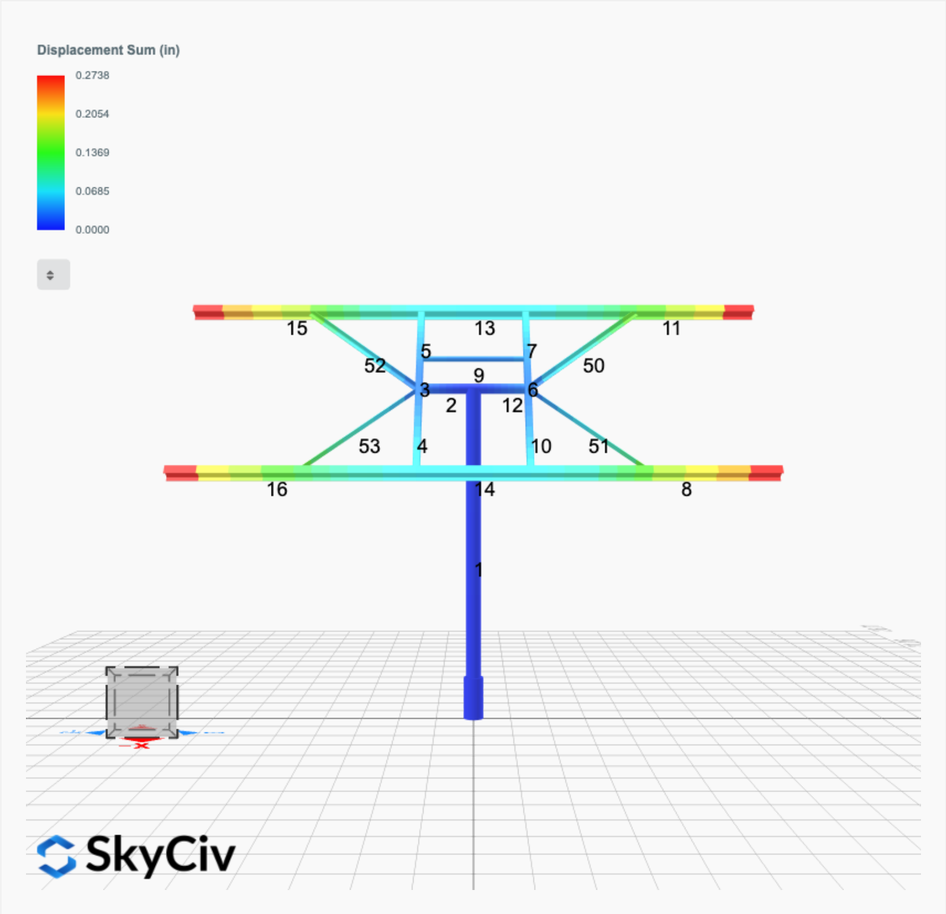
Rail Length: 17.333333333333332 ft
Additional Restraints Required: None
Tributary Width: 3.6750000000000003 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0236 kip/ft
Snow (Y): -0.0338 kip/ft
Wind uplift Case A: 0.0551 kip/ft
Wind downforce Case A: 0.0551 kip/ft
Dead (Panel load) (X): 0.0094 kip/ft
Dead (Panel load) (Y): -0.0135 kip/ft

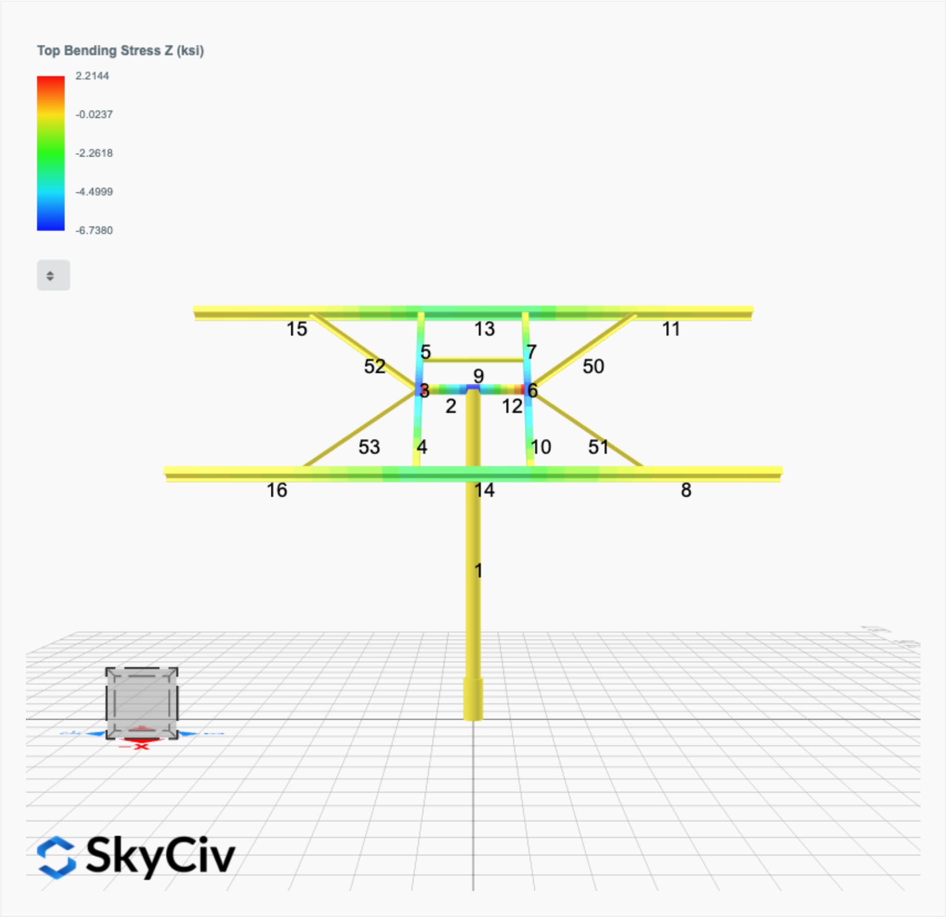


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	25.72429634	0.746	PASS
Material Yield	34.5	25.72429634	0.746	PASS
Material Strength	37	25.72429634	0.695	PASS



FEM Results (Envelope Worst Case for each member)

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-0.0237
-2.2618
-4.4999
-6.7380
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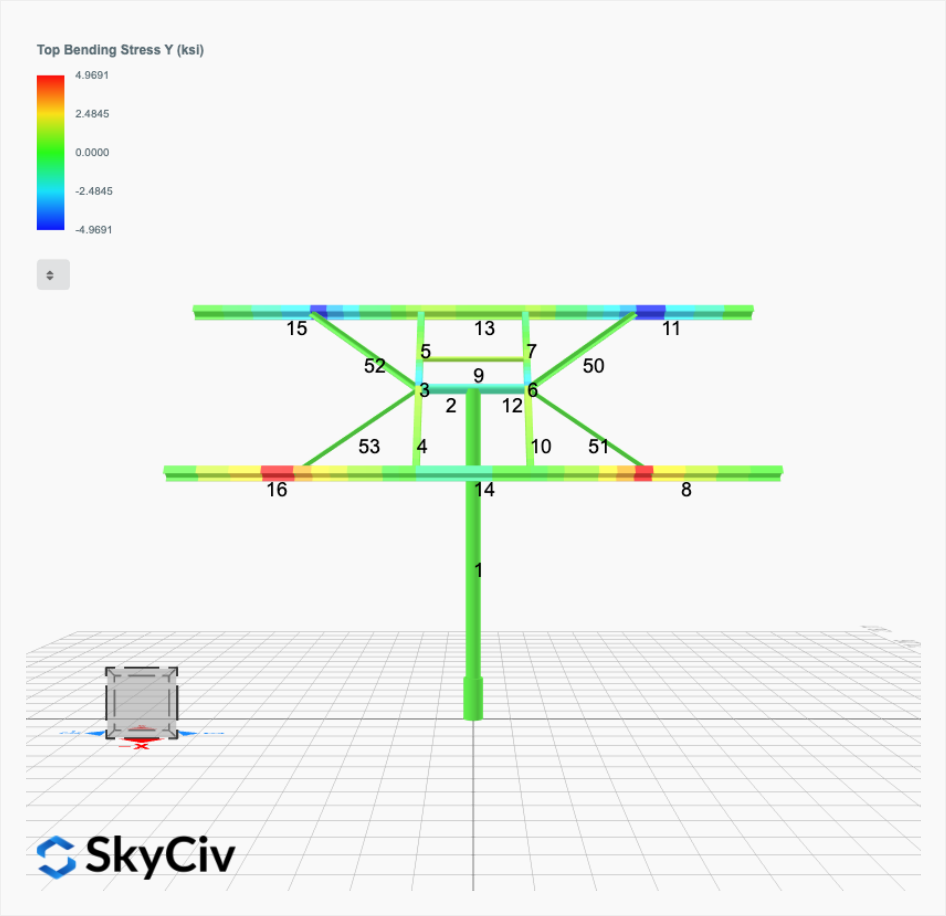
SkyCiv


Top Bending Stress Y (ksi)
4.9691
2.4845
0.0000
-2.4845
-4.9691
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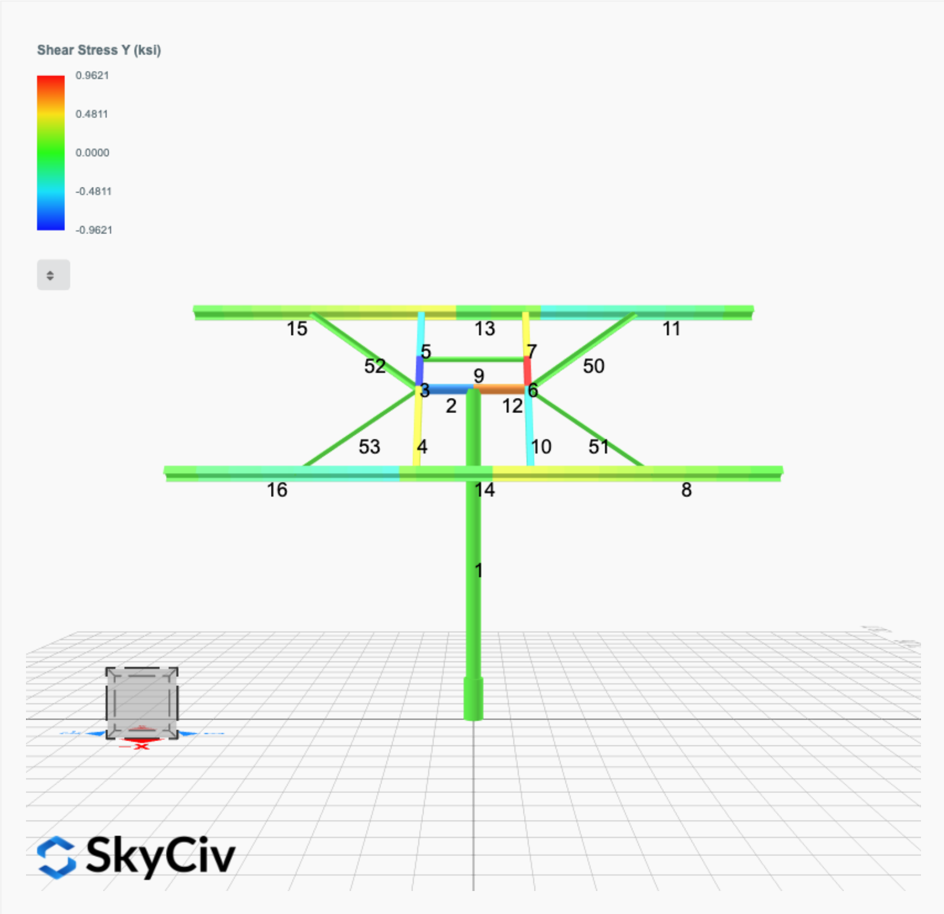
SkyCiv


Shear Stress Y (ksi)
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0.4811
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-0.4811
-0.9621
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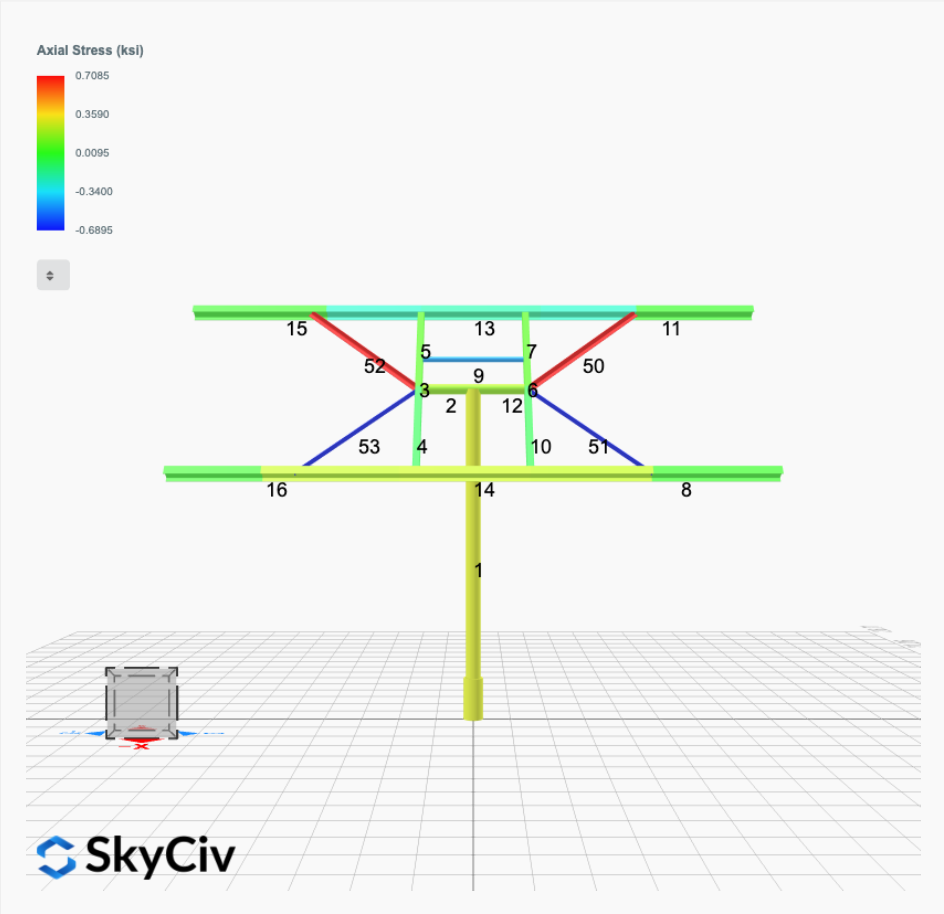
SkyCiv


Axial Stress (ksi)
0.7085
0.3590
0.0095
-0.3400
-0.6895
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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	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 2. D + L	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 3. D + (S or Lr or R)	-0.0000	5.0632	-0.0000	-0.0000	-0.0000	0.0416
ULS: 3. D + (S or Lr or R)	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.4638	-0.0000	-0.0000	-0.0000	0.0368
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 5b. D + 0.7E	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.4638	-0.0000	-0.0000	-0.0000	0.0368
ULS: 8. 0.6D + 0.7E	-0.0000	1.5994	-0.0000	-0.0000	-0.0000	0.0134
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8160	4.6375	-0.0000	-0.0000	-0.0000	34.4739
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8160	0.6938	-0.0000	-0.0000	-0.0000	-33.6805
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1120	5.9427	-0.0000	-0.0000	-0.0000	25.8755
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	4.4638	-0.0000	-0.0000	-0.0000	0.0368
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1120	2.9849	-0.0000	-0.0000	-0.0000	-25.2403
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	4.4638	-0.0000	-0.0000	-0.0000	0.0368
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1120	4.1445	-0.0000	-0.0000	-0.0000	25.8610
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1120	1.1868	-0.0000	-0.0000	-0.0000	-25.2548
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.6657	-0.0000	-0.0000	-0.0000	0.0224
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8160	3.5712	-0.0000	-0.0000	-0.0000	34.4650
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	1.5994	-0.0000	-0.0000	-0.0000	0.0134
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8160	-0.3724	-0.0000	-0.0000	-0.0000	-33.6894
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	1.5994	-0.0000	-0.0000	-0.0000	0.0134

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	8.6780
Shear X	-4.6934
Shear Z	0.0000
Moment X	-0.0018
Moment Y (Twist)	0.0007
Moment Z	58.4890

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	5.9427
Shear X	-2.8160
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	34.4739

Project Details

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

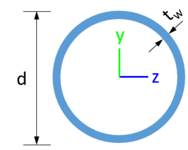
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

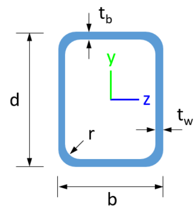
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

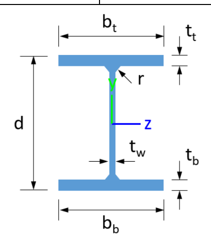
Section Dimensions



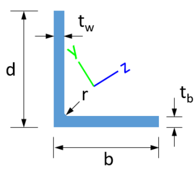
ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
9	8in Pipe Sch 40	8.63	0.32					



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
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	9	25.41	25.41	12.10	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	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	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
8	18	7.00	7.00	7.00	2.33,2.32,2.33,2.31,2.32,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.31,2.33,2.32,2.31,2.27,2.31,2.32,2.33,2.31,2.33,2.32,2.33,2.31,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.32,2.33,2.33,2.32,2.28,2.32,2.32,2.33,2.31,2.33,2.32,2.33,2.32,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
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15	18	7.00	7.00	7.00	2.33,2.32,2.33,2.32,2.33,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.32,2.33,2.33,2.32,2.28,2.32,2.32,2.33,2.31,2.33,2.32,2.33,2.32,2.33	300	200	1
16	18	7.00	7.00	7.00	2.33,2.32,2.33,2.31,2.32,2.33,2.32,2.32,2.32,2.32,2.32,2.33,2.31,2.33,2.32,2.31,2.27,2.31,2.32,2.33,2.31,2.33,2.32,2.33,2.31,2.33	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 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	171.95	83.29	83.29	113.39	113.39
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	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	72.01	10.99	4.60	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	98.23	17.96	6.45	30.09	45.74
14	120.60	98.23	17.96	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	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	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	0.050	0.702	0.000	0.041	0.000	0.725	#13	0.519	Not Required	Pass
2	0.008	0.404	0.311	0.096	0.055	0.691	#13	0.052	Not Required	Pass
3	0.003	0.687	0.305	0.069	0.073	0.878	#21	0.044	Not Required	Pass
4	0.003	0.678	0.101	0.068	0.006	0.740	#13	0.078	Not Required	Pass
5	0.002	0.426	0.068	0.069	0.016	0.467	#13	0.073	Not Required	Pass
6	0.003	0.687	0.305	0.069	0.073	0.878	#21	0.044	Not Required	Pass
7	0.002	0.426	0.068	0.069	0.016	0.467	#13	0.073	Not Required	Pass
8	0.020	0.194	0.190	0.043	0.012	0.266	#21	0.464	Not Required	Pass
9	0.031	0.056	0.082	0.001	0.000	0.147	#13	0.132	Not Required	Pass
10	0.003	0.678	0.101	0.068	0.006	0.740	#13	0.078	Not Required	Pass
11	0.011	0.195	0.190	0.044	0.013	0.261	#21	0.309	Not Required	Pass
12	0.008	0.404	0.311	0.096	0.055	0.691	#13	0.052	Not Required	Pass
13	0.011	0.397	0.032	0.054	0.006	0.419	#13	0.177	Not Required	Pass
14	0.014	0.399	0.033	0.054	0.006	0.405	#13	0.265	Not Required	Pass
15	0.011	0.195	0.190	0.044	0.013	0.261	#21	0.309	Not Required	Pass
16	0.020	0.194	0.190	0.043	0.012	0.266	#21	0.309	Not Required	Pass
50	0.206	0.009	0.006	0.002	0.002	0.217	#21	0.783	Not Required	Pass
51	0.041	0.003	0.014	0.001	0.001	0.034	#21	0.522	Not Required	Pass
52	0.206	0.009	0.006	0.002	0.002	0.217	#23	0.783	Not Required	Pass
53	0.041	0.003	0.014	0.001	0.001	0.034	#23	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

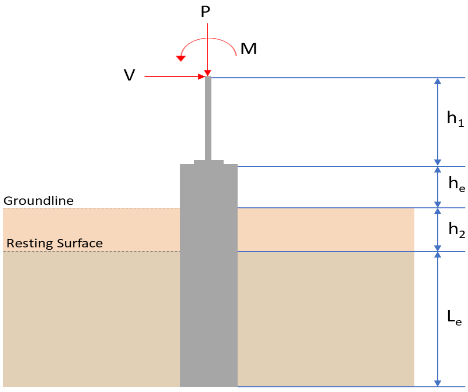
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 7\ \mathrm{ft}$ - 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)	5.943	8.678
V_x (kip)	-2.816	-4.693
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.002
M_z (kipft)	34.474	58.489

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,

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

$$H_o = \frac{(-2.816\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.44841\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.474 \text{ kipft}) + ((-2.816 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.4895 \text{ kipft/ft}$$

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:
$L_{e,x} = 6.4321 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(6.4321 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.432 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.432 \text{ ft})}{(7 \text{ ft})}$$

$$Ratio = 0.91886$$

Status: PASS
Ratio: 0.920

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

$$A = (48 \text{ in}) \times (48 \text{ in})$$

$$A = 16 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(5.943 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37144 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.37144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.18572$$

Status: PASS
Ratio: 0.190

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.75$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.44841\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.4895\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.4895\ \mathrm{kipft/ft}) \times (7\ \mathrm{ft})) + \left(3 \times (-0.44841\ \mathrm{kip/ft}) \times (7\ \mathrm{ft})^2\right)}{(6 \times (5.4895\ \mathrm{kipft/ft})) + (4 \times (-0.44841\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))}$$

$$a = 4.8277\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.4895\ \mathrm{kipft/ft})) + (3 \times (-0.44841\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]^{\,2}}{(7\ \mathrm{ft})^2 \times [(3 \times (5.4895\ \mathrm{kipft/ft})) + (2 \times (-0.44841\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}$$

$$p = 0.23624\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (5.4895\ \mathrm{kipft/ft})) + ((-0.44841\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}{(7\ \mathrm{ft})^2}$$

$$s = 0.96002\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.8277\ \mathrm{ft})}{2}$$

$$p_a = 0.36207\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23624\ \mathrm{kip/ft^2}\right)}{\left(0.36207\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65246$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7\ \mathrm{ft})$$

$$p_s = 1.05\ \mathrm{kip/ft^2}$$

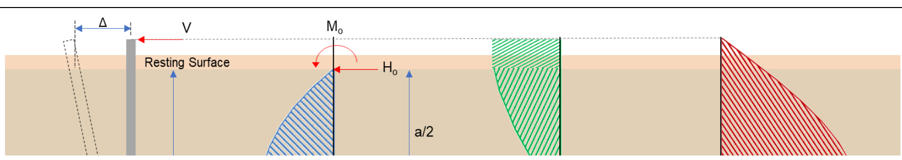
$Ratio$ - Lateral soil capacity

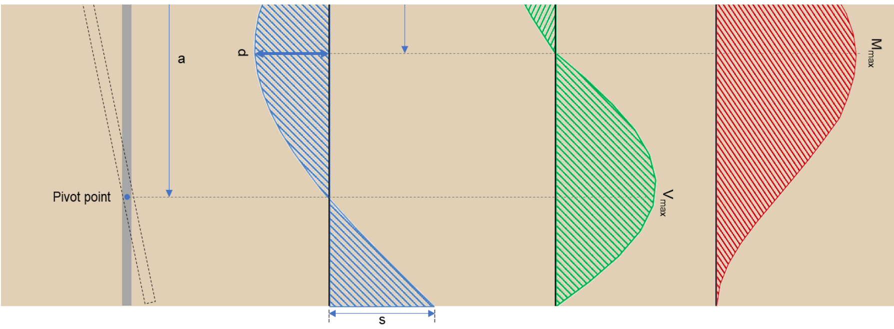
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.96002\ \mathrm{kip/ft^2}\right)}{\left(1.05\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9143$$

Status: **PASS**
Ratio: **0.910**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.693 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.74729 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(58.489 \text{ kipft}) + ((-4.693 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.3135 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.3135 \text{ kipft/ft})}{(-0.74729 \text{ kip/ft})}$$

$$E = 12.463 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.3135 \text{ kipft/ft}) \times (7 \text{ ft})) + \left(3 \times (-0.74729 \text{ kip/ft}) \times (7 \text{ ft})^2\right)}{(6 \times (9.3135 \text{ kipft/ft})) + (4 \times (-0.74729 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8256 \text{ ft}$$

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

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

$$V_{max} = ((-0.74729 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.463 \text{ ft})}{(7 \text{ ft})} + 3\right) \times \left(\frac{(4.8256 \text{ ft})}{(7 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.463 \text{ ft})}{(7 \text{ ft})} + 2\right) \times \left(\frac{(4.8256 \text{ ft})}{(7 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 11.389 \text{ kip}$$

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

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

$$M_{max} = ((-0.74729 \text{ kip/ft}) \times (48 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(12.463 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8256 \text{ ft})}{2 \times (7 \text{ ft})}\right) - \left[\left(\frac{4 \times (12.463 \text{ ft})}{(7 \text{ ft})} + 3\right) \times \left(\frac{(4.8256 \text{ ft})}{2 \times (7 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.463 \text{ ft})}{(7 \text{ ft})} + 2\right) \times \left(\frac{(4.8256 \text{ ft})}{2 \times (7 \text{ ft})}\right)^4\right]\right]$$

	$$M_{max} = 37.962 \text{ kipft}$$		
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $$H_o = \frac{V_z}{1.57\, b}$$ $$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$ $$H_o = 0 \text{ kip/ft}$$ M_o - Moment per length of pile, $$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$ $$M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$ $$M_o = 0.00031847 \text{ kipft/ft}$$ a - Distance from resting surface to pivot point, $$a = \frac{(4\, M_o\, L_e) + \left(3\, H_o\, L_e^2\right)}{(6\, M_o) + (4\, H_o\, L_e)}$$ $$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (7 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7 \text{ ft})^2\right)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7 \text{ ft}))}$$ $$a = 4.6667 \text{ ft}$$ V_{max} - Max shear force located at depth a, $$V_{max} = 12 \left(\frac{M_o\, b}{L_e}\right) \left(\frac{a}{L_e} - 1\right) \left(\frac{a}{L_e}\right)^2$$ $$V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(7 \text{ ft})}\right) \times \left(\frac{(4.6667 \text{ ft})}{(7 \text{ ft})} - 1\right) \times \left(\frac{(4.6667 \text{ ft})}{(7 \text{ ft})}\right)^2$$ $$V_{max} = 0.00032353 \text{ kip}$$ M_{max} - Max bending moment at depth a/2, $$M_{max} = (M_o\, b) \left[1 - \left(4\, \frac{a}{2\, L_e}^3\right) + \left(3\, \frac{a}{2\, L_e}^4\right)\right]$$ $$M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.6667 \text{ ft})}{2 \times (7 \text{ ft})}^3\right) + \left(3 \times \frac{(4.6667 \text{ ft})}{2 \times (7 \text{ ft})}^4\right)\right]$$ $$M_{max} = 0.0011323 \text{ kipft}$$		
Table 22.4.2.1 22.4.2.2, 10.6.1.1	**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, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, **Longitudinal reinforcement:** Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $$A_{st,required} = Min\left[\frac{\frac{P}{\phi\, \alpha} - \left(0.85\, f'_{ck}\, A_g\right)}{f_{yk} - \left(0.85\, f'_{ck}\right)}, (0.08\, A_g)\right]$$ $$A_{st,required} = Min\left[\frac{\frac{(8.678 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$ $$A_{st,required} = -84.308 \text{ in}^2$$ A_{min} - Governing minimum reinforcement area, $$A_{min} = Max\left[A_{st,required}, (0.0018\, A_g)\right]$$ $$A_{min} = Max\left[\left(-84.308 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$ $$A_{min} = 4.1472 \text{ in}^2$$ n_{rebar} - Required number of reinforcement		

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.678 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0032439$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80 \, D$$

$$d = 0.80 \times (48 \text{ in})$$

$$d = 38.4\ \text{in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,max} = 296.21\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 8.678\ \text{kip} \rightarrow 8678\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(8678\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.64\ \text{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.64\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.64\ \text{kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{s,a} = 737.28\ \text{kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.64\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 110.85\ \text{kip}$$

Considering x-direction:

$V_{max} = 11.389\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(11.389\ \text{kip})}{(110.85\ \text{kip})}$$

$$Ratio = 0.10275$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

$$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$

$$S_m = 18432\ \text{in}^3$$

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

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$

$$\phi M_{n,1} = 249.600kipft$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(18432\ \text{in}^3\right)$$

$$\phi M_{n,2} = 2121.6\ \text{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(249.6\ \text{kipft}), (2121.6\ \text{kipft})]$$

$$\phi M_n = 249.6\ \text{kipft}$$

Considering x-direction:

$M_{max} = 37.962\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(37.962\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.15209$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$M_{max} = 0.0011323\ \text{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

		ϕM_n $$Ratio = \frac{(0.0011323\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 4.5366 \times 10^{-6}$$	Status: **PASS** Ratio: **0.000**