Project Details

Project Name: MTSOLAR_FB17714DLA0F

Location: 40394 Schlies Rd, Highbridge, WI 54846, USA

Unique ID: 1P-0-8TOP-SD-84-L-4Hx3W-I6G4

Dealer: ______________________________

Date: Fri Aug 16 2024

Number of Modules: 12

Number of Poles: 1

Date Sold:

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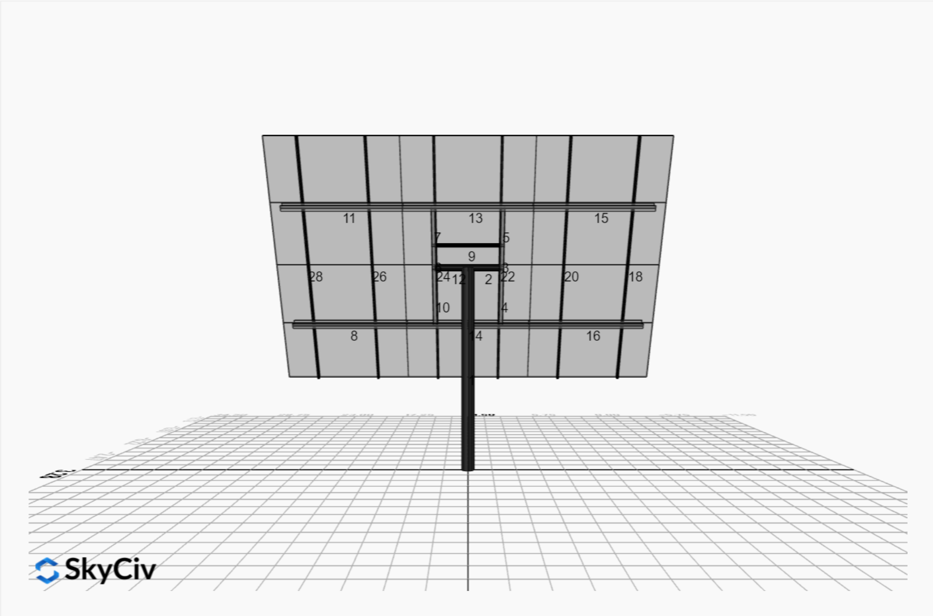

Array Dimensions N/S	15.50 ft
Array Dimensions E/W	23.00 ft
Winter Tilt Angle	65
Front Edge Clearance	5 ft

MT Solar Bill of Materials (1P-0-8TOP-SD-84-L-4Hx3W-I6G4)

Part	Short Description	BOM Qty
MTS-PC-8	8IN 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	3

Rail Bill of Materials

Part	Qty
Rails (184in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12
Ground Lug	3

Site Details:

Site Address: 40394 Schlies Rd, Highbridge, WI 54846, USA

Array Specification

Duty Classification:	SD
Module Width:	46.00 in
Module Length:	91.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	65
Front Edge Clearance:	5
Total Array Height at Tilt:	19.05 ft
Total Frame Length:	21.50 ft
Frame Weight:	1324 lbs
Array Dimensions N/S:	15.50 ft
Array Dimensions E/W:	23.00 ft
Rail Length:	186.00 in
Rail Spacing:	3.83 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	12.02 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: 7.75 ft
Foundation Volume:	4.593 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	40394 Schlies Rd, Highbridge, WI 54846, USA
Wind Speed:	99 mph
Snow Load:	60 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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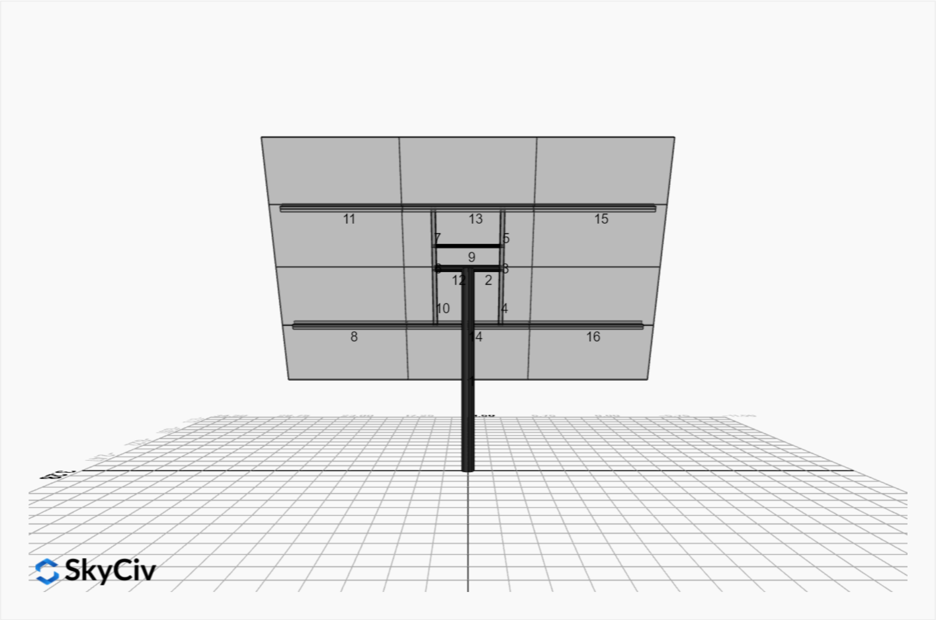
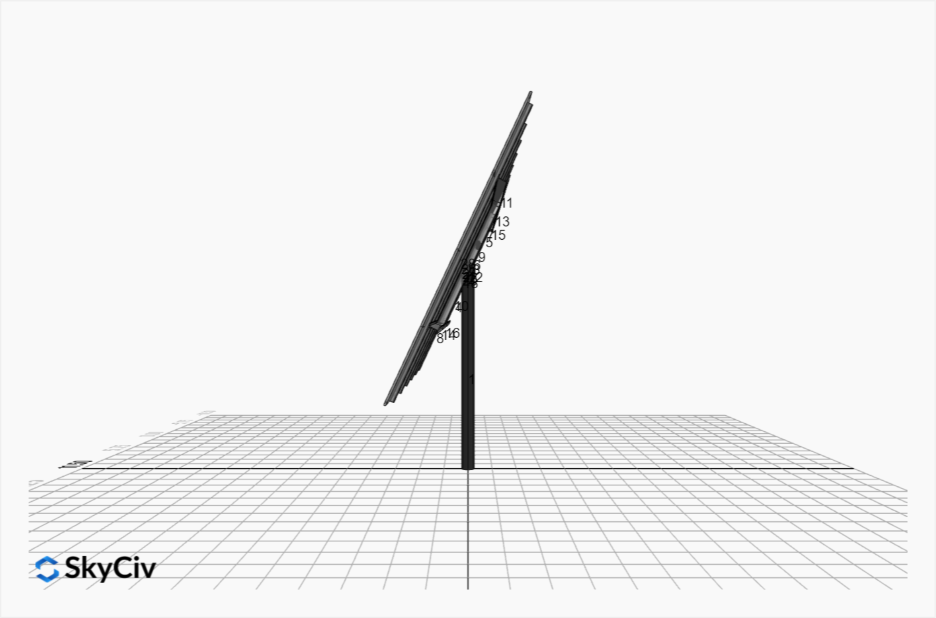
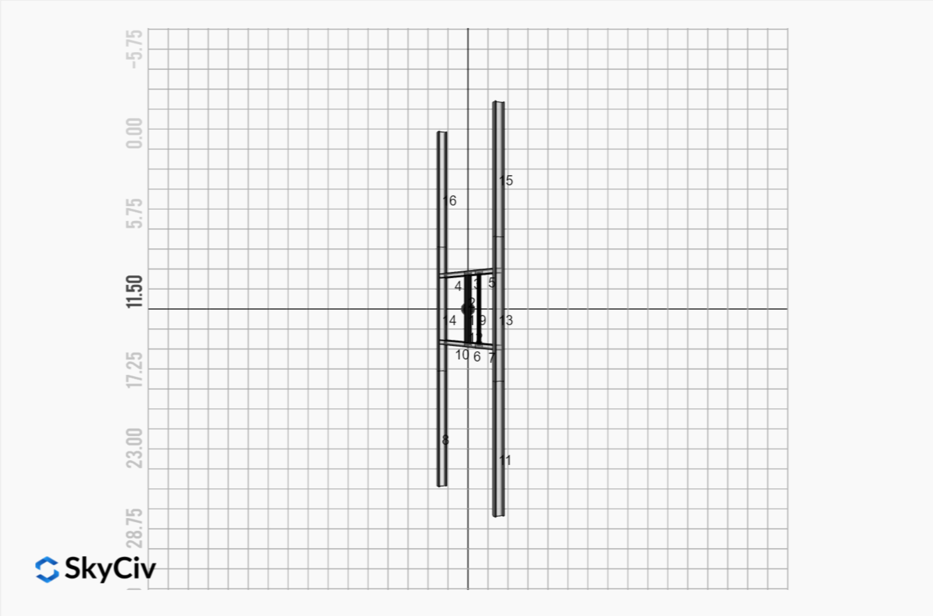
Design Notes:

- 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)
- Foundation Design and Sizing is approximate only



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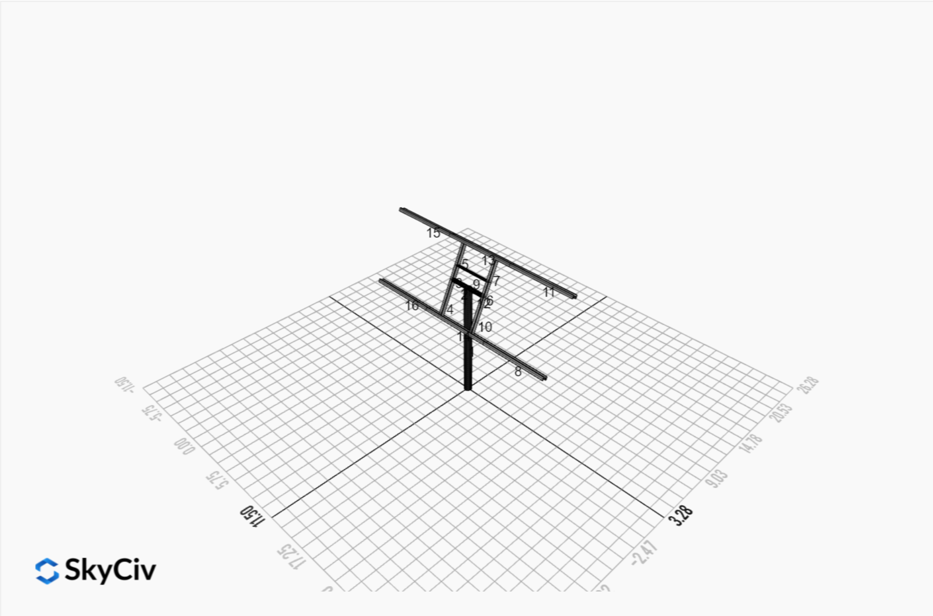
5.75
11.50
17.25
23.00
28.75
15
16
13
10
6
7
8
11
SkyCiv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13
7
9
16
4
10
11
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3.28
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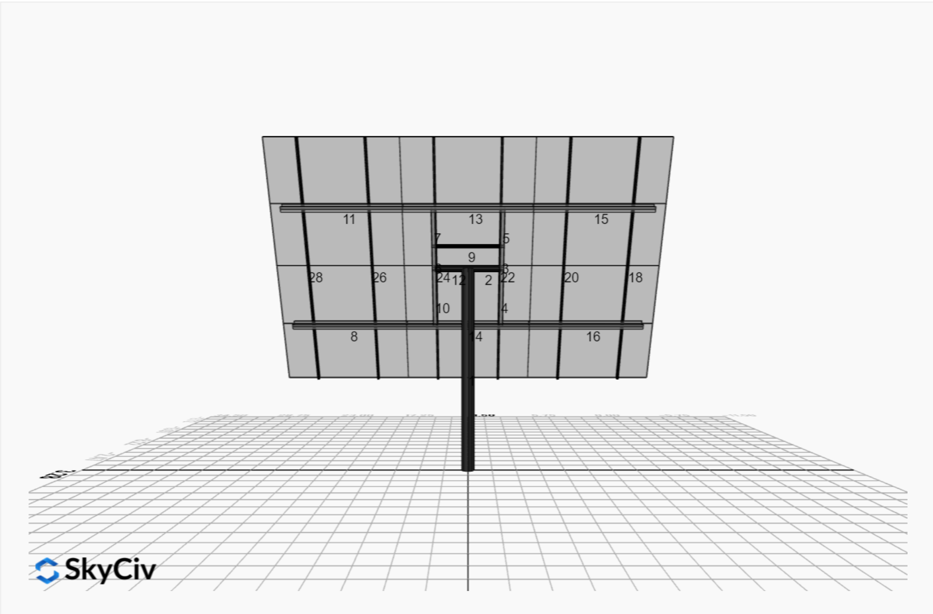
9.03
14.78
20.53
SkyCiv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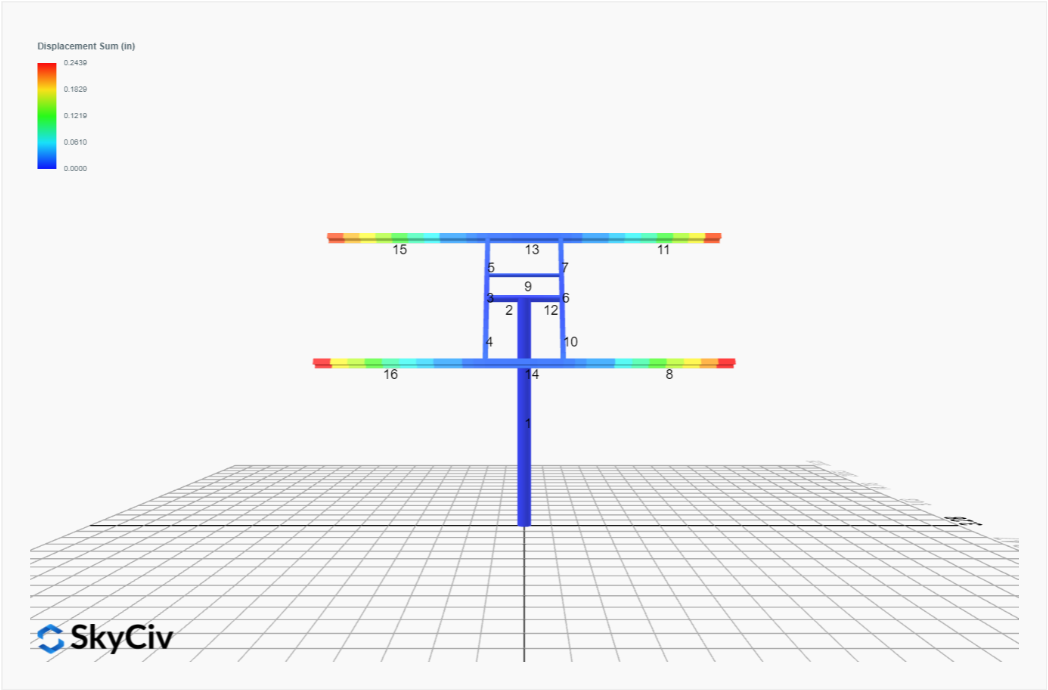
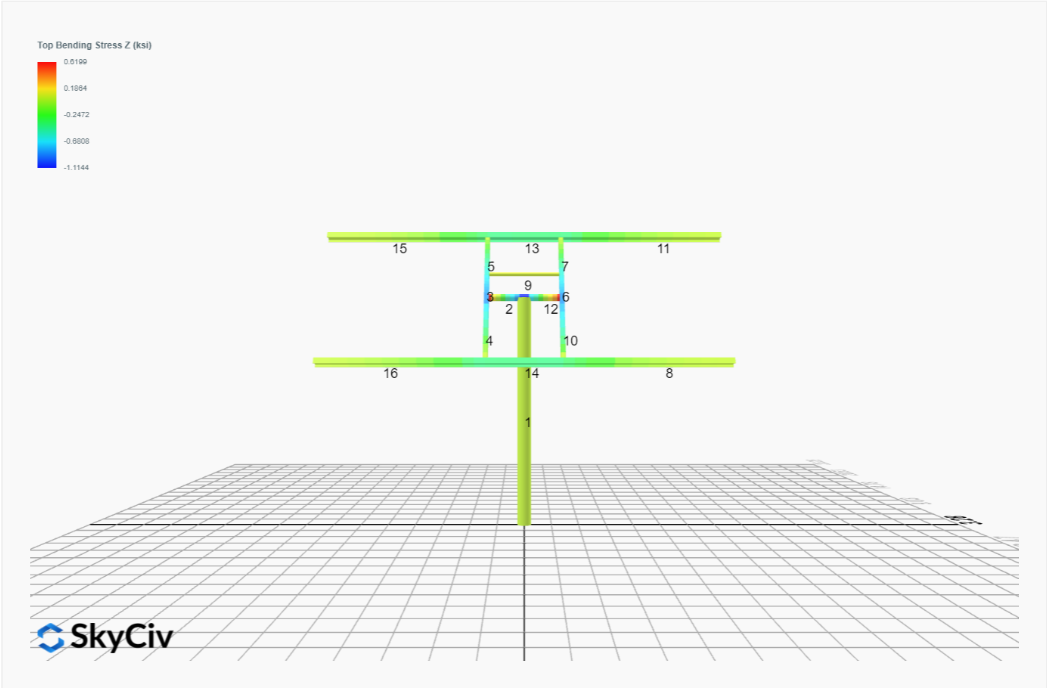
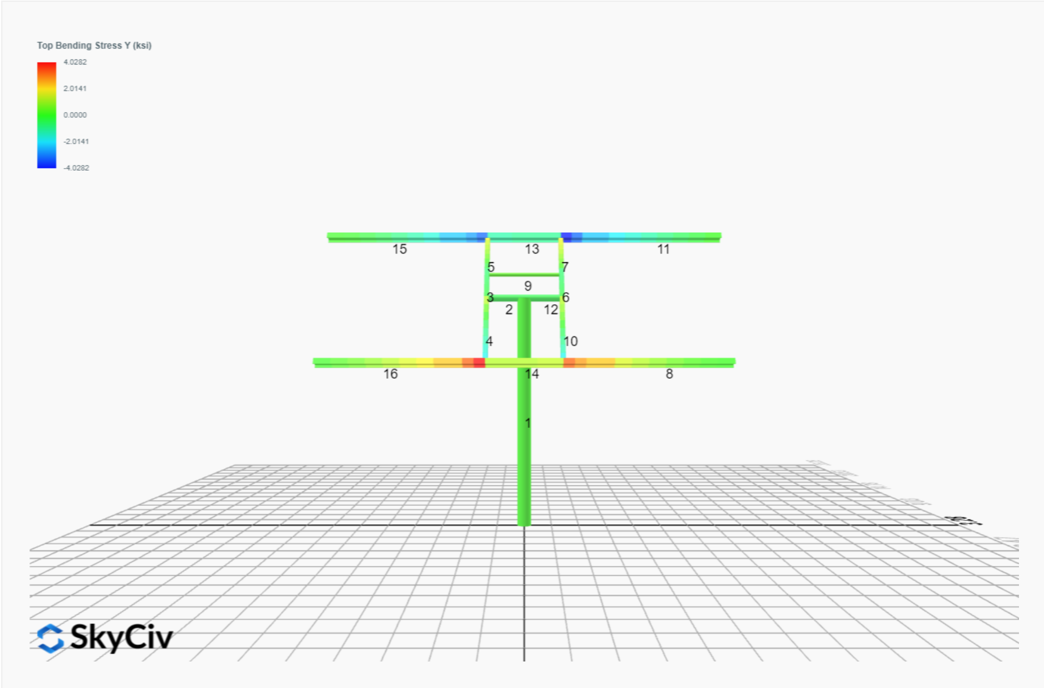
13
15
5
9
28
26
24
12
2
22
20
18
10
4
8
16
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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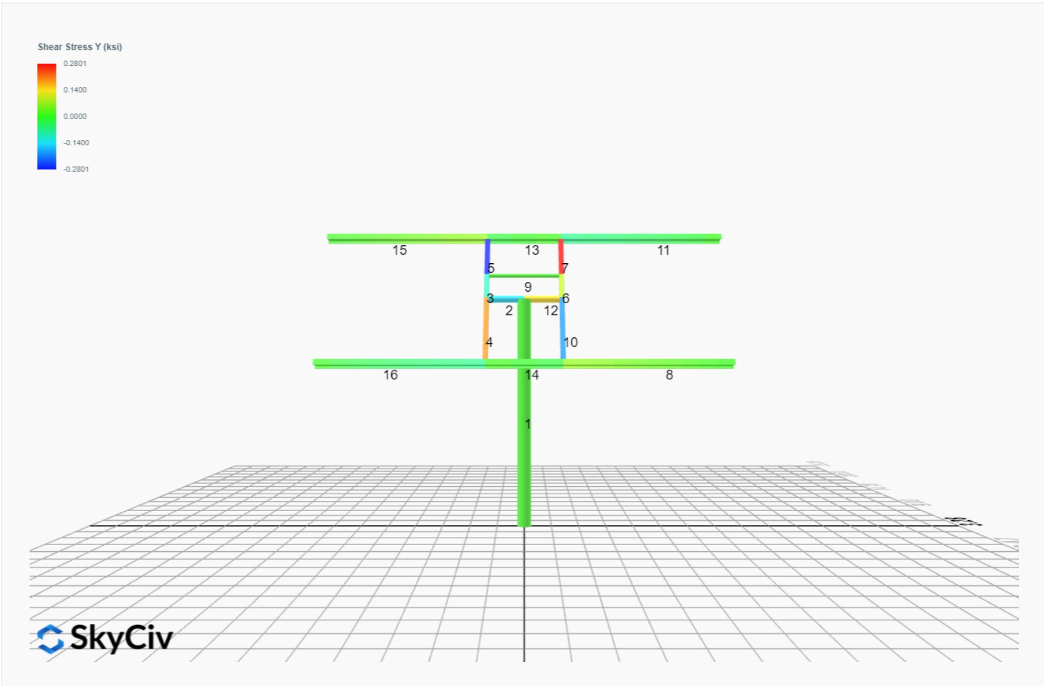
2.0141
0.0000
-2.0141
-4.0282
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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0.1400
0.0000
-0.1400
-0.2801
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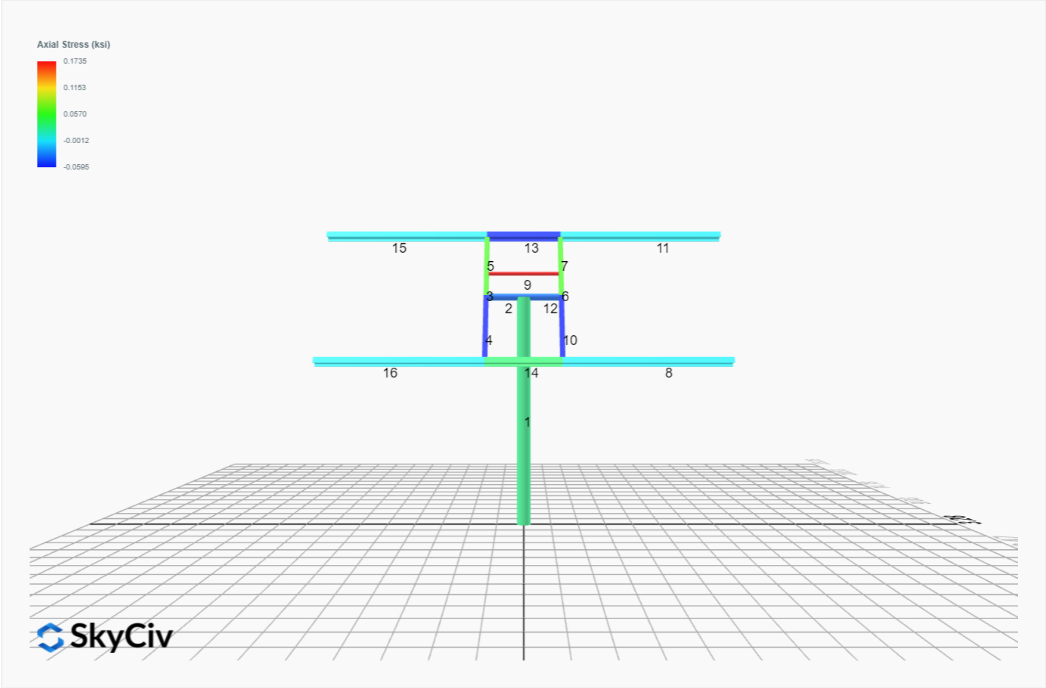
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv



Axial Stress (ksi)
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0.1153
0.0570
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-0.0595
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
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	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 2. D + L	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 3. D + (S or Lr or R)	0.0000	3.1331	0.0000	0.0000	-0.0000	0.0129
ULS: 3. D + (S or Lr or R)	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.0169	0.0000	0.0000	-0.0000	0.0129
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 5b. D + 0.7E	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.0169	0.0000	0.0000	-0.0000	0.0129
ULS: 8. 0.6D + 0.7E	0.0000	1.6011	0.0000	0.0000	-0.0000	0.0076
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.3882	4.7147	0.0000	0.0000	-0.0000	53.1872
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.3882	0.6222	0.0000	0.0000	-0.0000	-52.3472
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2912	4.5516	0.0000	0.0000	-0.0000	39.8938
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.0169	0.0000	0.0000	-0.0000	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2912	1.4823	0.0000	0.0000	-0.0000	-39.2570
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.0169	0.0000	0.0000	-0.0000	0.0129
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2912	4.2032	0.0000	0.0000	-0.0000	39.8936
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2912	1.1338	0.0000	0.0000	-0.0000	-39.2573
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.6685	0.0000	0.0000	-0.0000	0.0126
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.3882	3.6474	0.0000	0.0000	-0.0000	53.1822
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.6011	0.0000	0.0000	-0.0000	0.0076
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.3882	-0.4452	0.0000	0.0000	-0.0000	-52.3523
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.6011	0.0000	0.0000	-0.0000	0.0076

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	6.8449
Shear X	-7.3137
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	89.4222

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	4.7147
Shear X	-4.3882
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	53.1872

Project Details

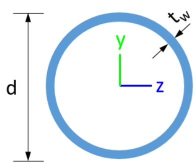
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	MTSOLAR_FB17714DLA0F
Unit System:	imperial

Design Input Information

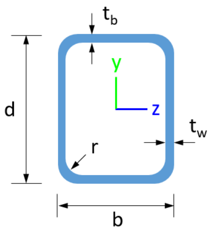
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

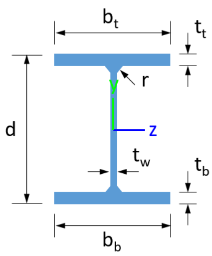
Section Dimensions



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					
10	8in Pipe Sch 80	8.63	0.50					



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



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)

1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	10	25.25	25.25	12.02	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
8	18	14.70	14.70	7.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
11	18	14.70	14.70	7.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	300	200	1
14	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	14.70	14.70	7.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	14.70	14.70	7.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1

Member Design Capacity

Member ID	Φ_tP_n (kip)	Φ_cP_n (kip)	Φ_bM_{zn} (k-ft)	Φ_bM_{yn} (k-ft)	Φ_vV_{yn} (kip)	Φ_vV_{zn} (kip)
1	574.32	255.38	123.94	123.94	172.30	172.30
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	15.97	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	6.26	29.14	16.61
11	120.60	15.97	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	17.91	6.45	30.09	45.74

14	120.60	84.03	17.90	6.45	30.09	45.74
15	120.60	15.97	23.36	6.45	30.09	45.74
16	120.60	15.97	23.36	6.45	30.09	45.74

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.027	0.722	0.000	0.042	0.000	0.735	#13	0.526	Not Required	Pass
2	0.007	0.312	0.455	0.073	0.085	0.770	#13	0.034	Not Required	Pass
3	0.010	0.793	0.077	0.079	0.011	0.827	#13	0.044	Not Required	Pass
4	0.009	0.789	0.288	0.079	0.046	0.932	#13	0.078	Not Required	Pass
5	0.010	0.493	0.307	0.079	0.063	0.566	#13	0.073	Not Required	Pass
6	0.010	0.793	0.077	0.079	0.011	0.827	#13	0.044	Not Required	Pass
7	0.010	0.493	0.307	0.079	0.063	0.566	#13	0.073	Not Required	Pass
8	0.000	0.224	0.252	0.050	0.010	0.435	#13	Not Required	Not Required	Pass
9	0.029	0.047	0.107	0.001	0.000	0.156	#13	0.198	Not Required	Pass
10	0.009	0.789	0.288	0.079	0.046	0.932	#13	0.078	Not Required	Pass
11	0.000	0.224	0.252	0.050	0.010	0.435	#13	Not Required	Not Required	Pass
12	0.007	0.312	0.455	0.073	0.085	0.770	#13	0.034	Not Required	Pass
13	0.009	0.456	0.393	0.062	0.013	0.785	#13	0.177	Not Required	Pass
14	0.009	0.463	0.393	0.062	0.013	0.786	#13	0.177	Not Required	Pass
15	0.000	0.224	0.252	0.050	0.010	0.435	#13	Not Required	Not Required	Pass
16	0.000	0.224	0.252	0.050	0.010	0.435	#13	Not Required	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

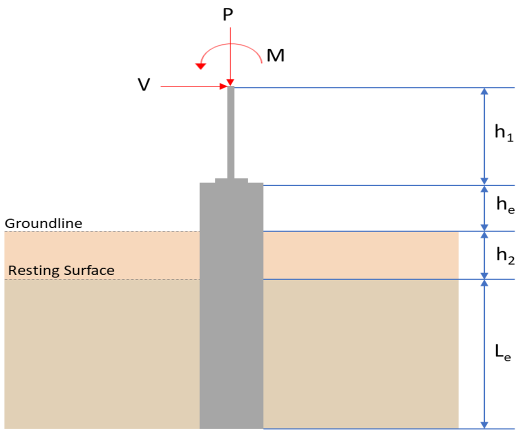
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48\ \mathrm{in}$ - Pile width

$D = 48\ \mathrm{in}$ - Pile depth

$L = 7.75\ \mathrm{ft}$ - 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)	4.715	6.845
V_x (kip)	-4.388	-7.314
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	53.187	89.422

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,

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

$$H_o = \frac{(-4.388\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.69873\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.187\text{ kipft}) + ((-4.388\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 8.4693\text{ kipft/ft}$$

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:
$L_{e,x} = 7.2124\text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(7.2124\text{ ft}), (0\text{ ft})]$$

$$L_{e,req} = 7.212\text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75\text{ ft}) - (0\text{ ft}) - (0\text{ ft})$$

$$L_e = 7.75\text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.212\text{ ft})}{(7.75\text{ ft})}$$

$$Ratio = 0.93058$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.715\text{ kip})}{(16\text{ ft}^2)}$$

$$q = 0.29469\text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.29469\text{ kip/ft}^2)}{(2000\text{ psf})}$$

$$Ratio = 0.14734$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.9375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.69873 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 8.4693 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4693 \text{ kipft/ft}) \times (7.75 \text{ ft})) + \left(3 \times (-0.69873 \text{ kip/ft}) \times (7.75 \text{ ft})^2\right)}{(6 \times (8.4693 \text{ kipft/ft})) + (4 \times (-0.69873 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.3597 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.4693 \text{ kipft/ft})) + (3 \times (-0.69873 \text{ kip/ft}) \times (7.75 \text{ ft}))]^{\ 2}}{(7.75 \text{ ft})^2 \times [(3 \times (8.4693 \text{ kipft/ft})) + (2 \times (-0.69873 \text{ kip/ft}) \times (7.75 \text{ ft}))]}$$

$$p = 0.26629 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (8.4693 \text{ kipft/ft})) + ((-0.69873 \text{ kip/ft}) \times (7.75 \text{ ft}))]}{(7.75 \text{ ft})^2}$$

$$s = 1.1511 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3597 \text{ ft})}{2}$$

$$p_a = 0.40198 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.26629 \text{ kip/ft}^2\right)}{\left(0.40198 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.66246$$

Status: **PASS**
Ratio: **0.660**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.75 \text{ ft})$$

$$p_s = 1.1625 \text{ kip/ft}^2$$

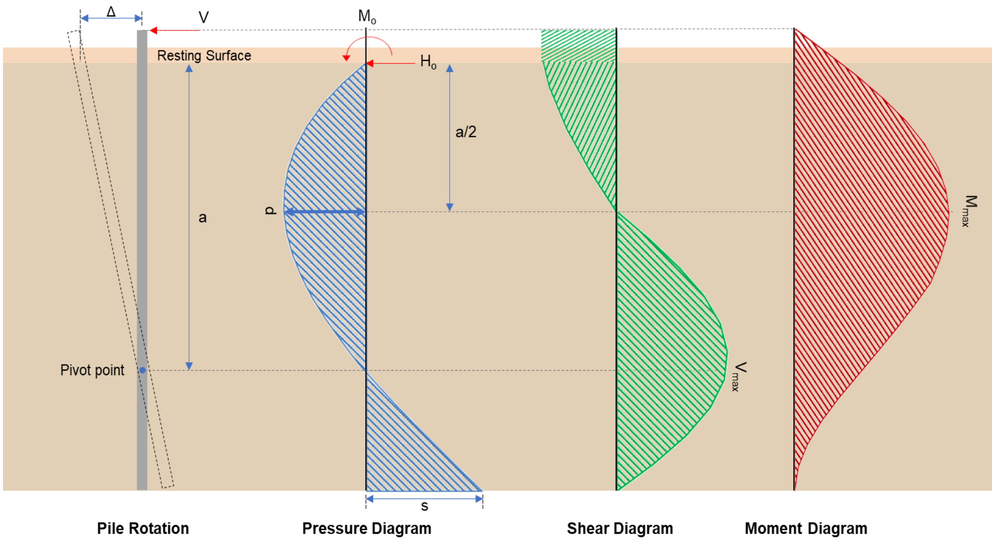
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.1511 \text{ kip/ft}^2\right)}{\left(1.1625 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.99023$$

Status: PASS
Ratio: 0.990



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.314\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -1.1646\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(89.422\ \text{kipft}) + ((-7.314\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 14.239\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.239\ \text{kipft/ft})}{(-1.1646\ \text{kip/ft})}$$

$$E = 12.226\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.239\ \text{kipft/ft}) \times (7.75\ \text{ft})) + \left(3 \times (-1.1646\ \text{kip/ft}) \times (7.75\ \text{ft})^2\right)}{(6 \times (14.239\ \text{kipft/ft})) + (4 \times (-1.1646\ \text{kip/ft}) \times (7.75\ \text{ft}))}$$

$$a = 5.3585\ \text{ft}$$

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

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

$$V_{max} = ((-1.1646\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.226\ \text{ft})}{(7.75\ \text{ft})} + 3\right) \times \left(\frac{(5.3585\ \text{ft})}{(7.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.226\ \text{ft})}{(7.75\ \text{ft})} + 2\right) \times \left(\frac{(5.3585\ \text{ft})}{(7.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 16.076\ \text{kip}$$

$$V_{max} = 16.076 \text{ kip}$$

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

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

$$M_{max} = ((-1.1646 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(12.226 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3585 \text{ ft})}{2 \times (7.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.226 \text{ ft})}{(7.75 \text{ ft})} + 3 \right) \times \left(\frac{(5.3585 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.226 \text{ ft})}{(7.75 \text{ ft})} + 2 \right) \times \left(\frac{(5.3585 \text{ ft})}{2 \times (7.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 59.022 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(6.845 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.369 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.369 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.845 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0025587$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.845 \text{ kip} \rightarrow 6845 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(6845\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 119.4\ \mathrm{kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (119.4\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 119.4\ \mathrm{kip}$$

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)

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

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

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

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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.4\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 110.69\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 16.076\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(16.076\ \mathrm{kip})}{(110.69\ \mathrm{kip})}$$

$$Ratio = 0.14524$$

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

	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:** $M_{max} = 59.022\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(59.022\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.23647$$	Status: **PASS** Ratio: **0.240**