Your Project Calculations

Project Name: MTSOLAR_5ECA5L856AE5

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MTSOLAR_5ECA5L856AE5&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=G167qzZJE7JEircyroNGyxUCMWITXdD3CfWa1h7Z6z7HQGPB14SdZxIb8nusZqmx

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-45-L-4Hx3W-BL5H
Duty Classification:	SD
Module Width:	40.00 in
Module Length:	65.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	0
Front Edge Clearance:	12
Total Array Height at Tilt:	12.00 ft
Total Frame Length:	15.00 ft
Frame Weight:	650 lbs
Array Dimensions N/S:	13.50 ft
Array Dimensions E/W:	16.50 ft
Rail Length:	162.00 in
Rail Spacing:	2.71 ft
Rail Check:	Not Checked

Support Specifications

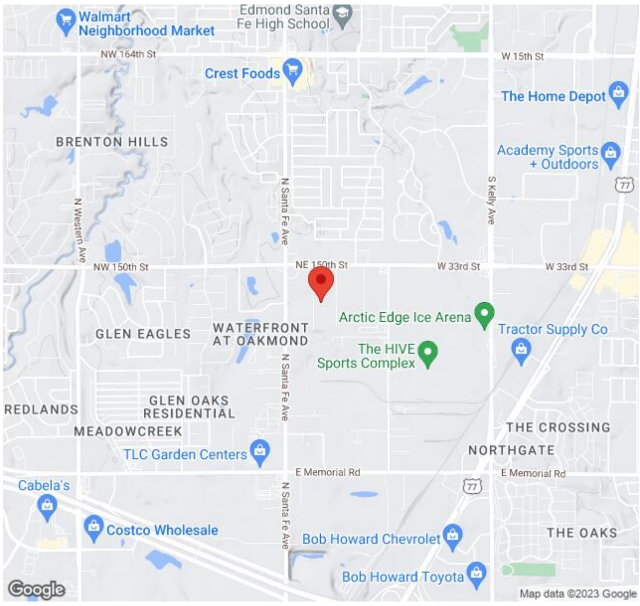
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	12.00 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 3.75 ft
Foundation Volume:	2.222 y^3
Foundation Result:	PASSED
Mount Twist:	0.000000 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	14800 Santa Fe Crossing Dr, Edmond, OK 73013, USA
Wind Speed:	100 mph
Snow Load:	10 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.011048 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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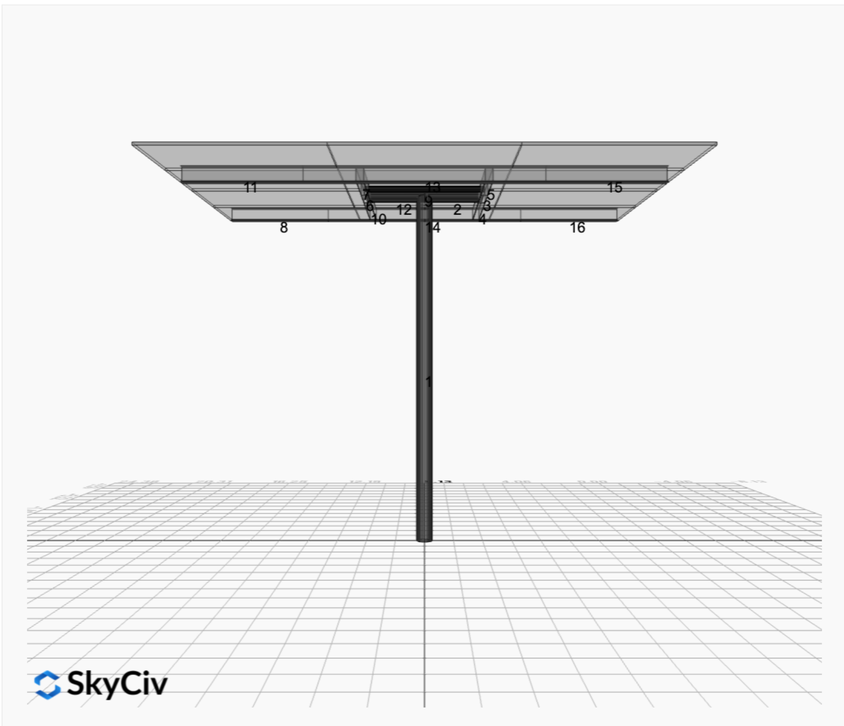
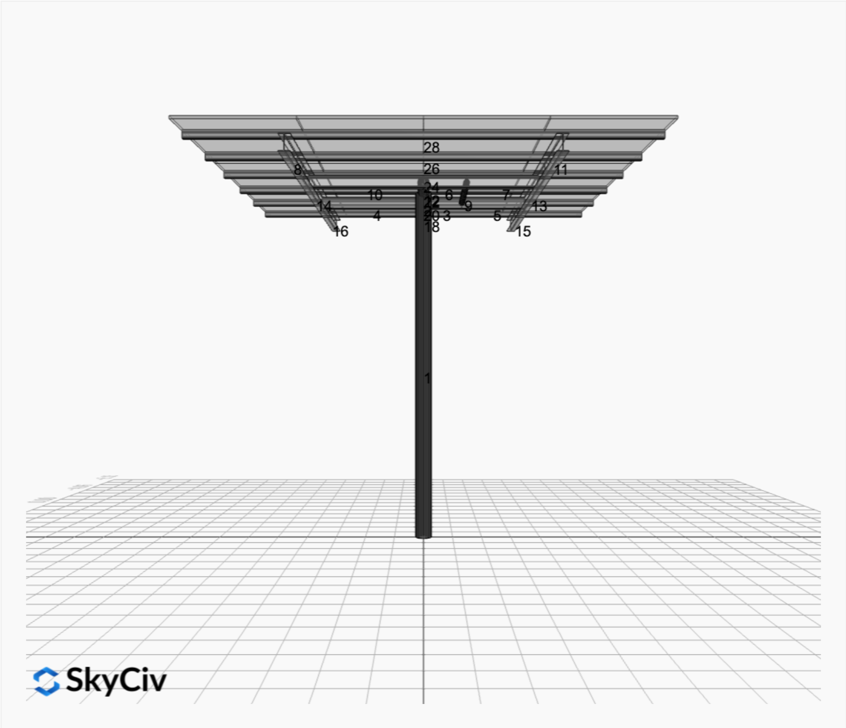
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

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15
18
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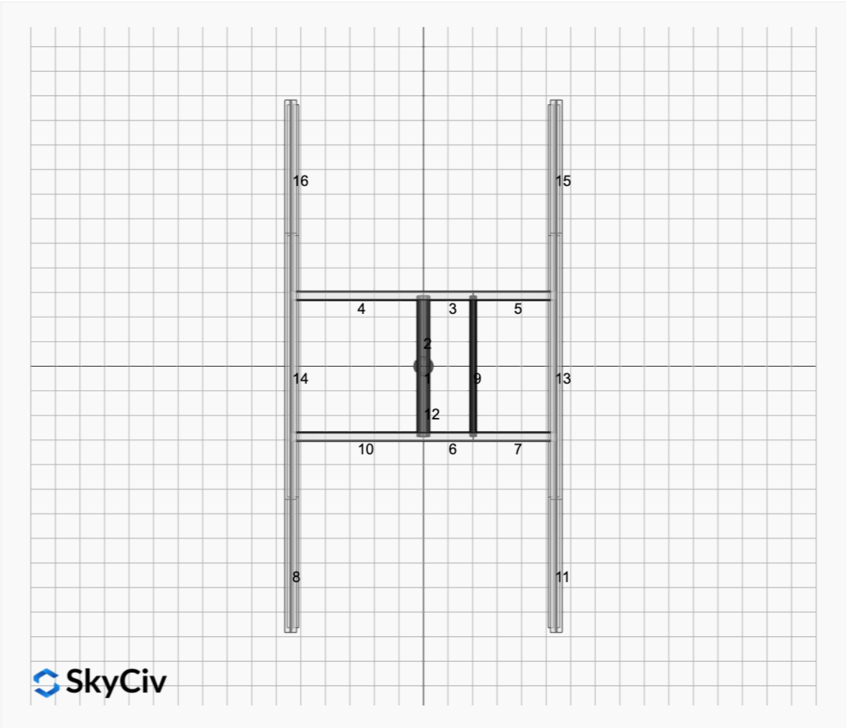
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14
1
9
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12
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6
7
8
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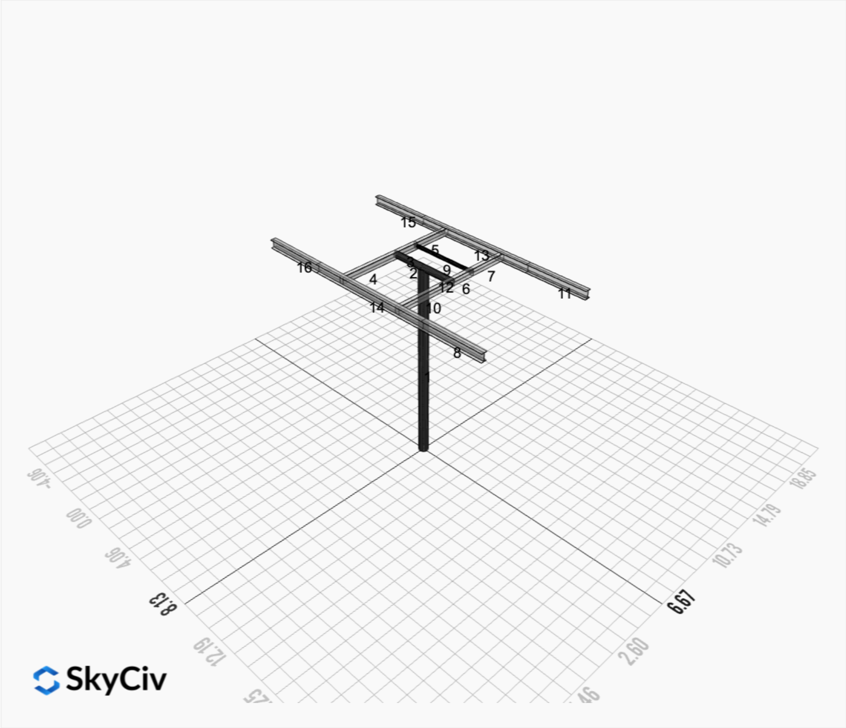
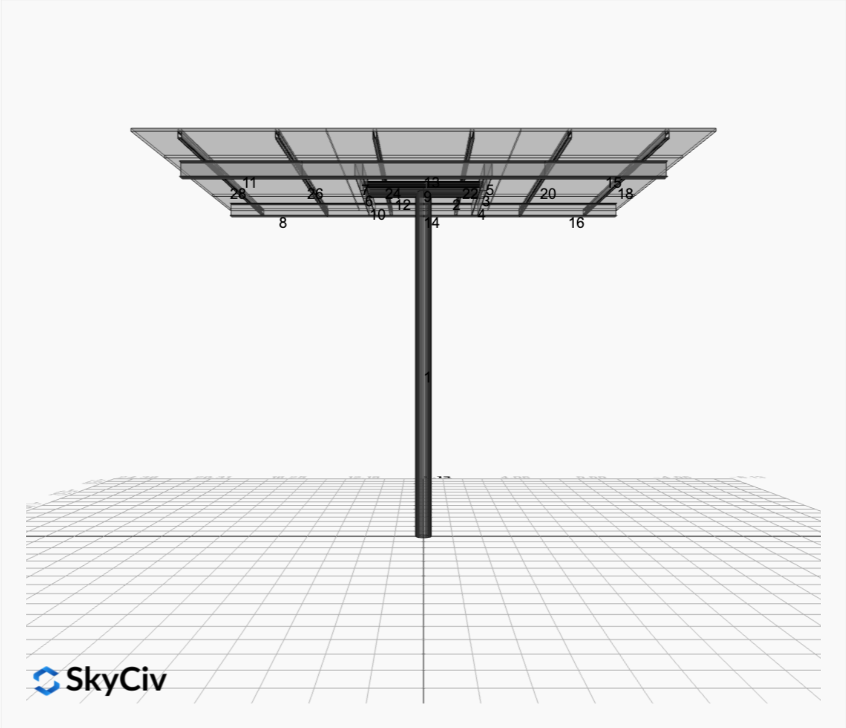
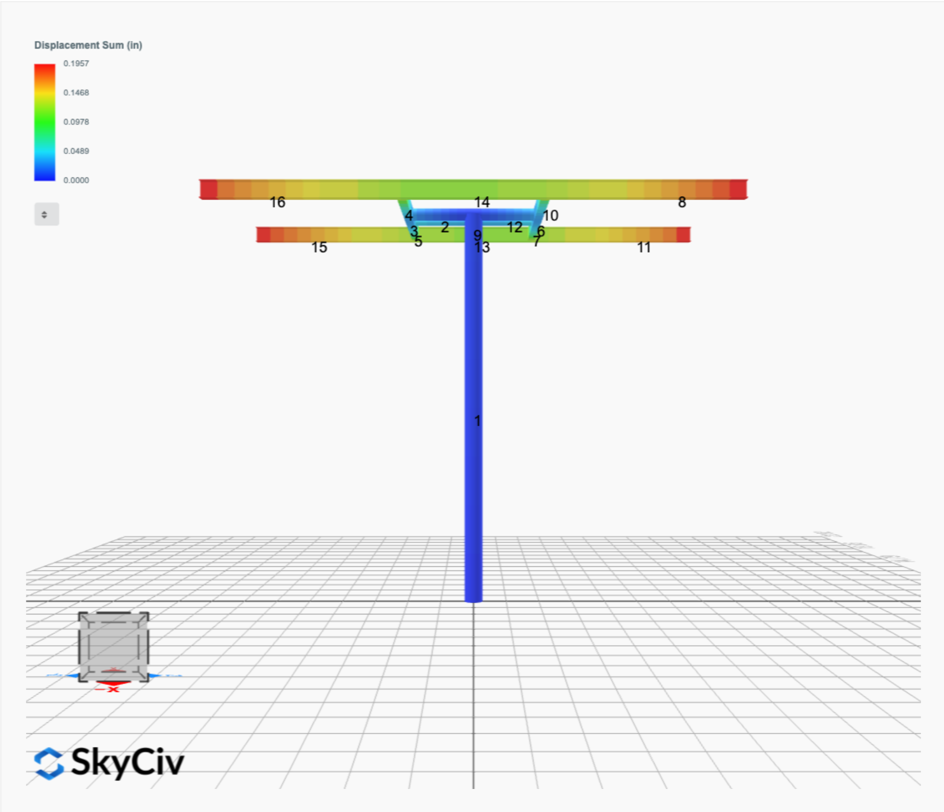
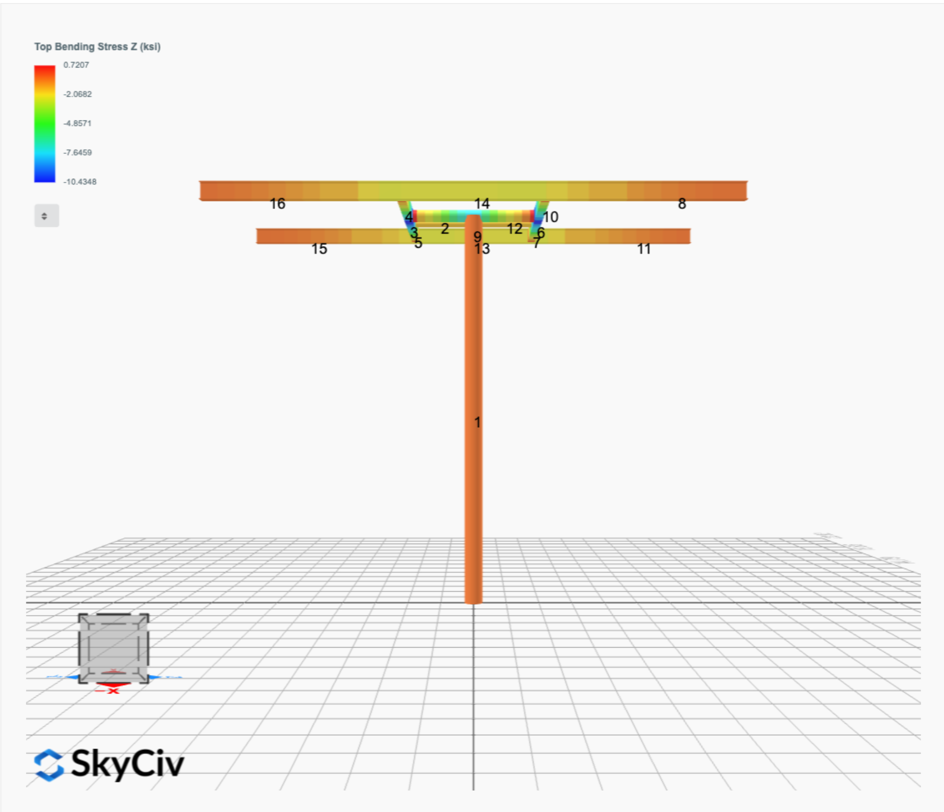
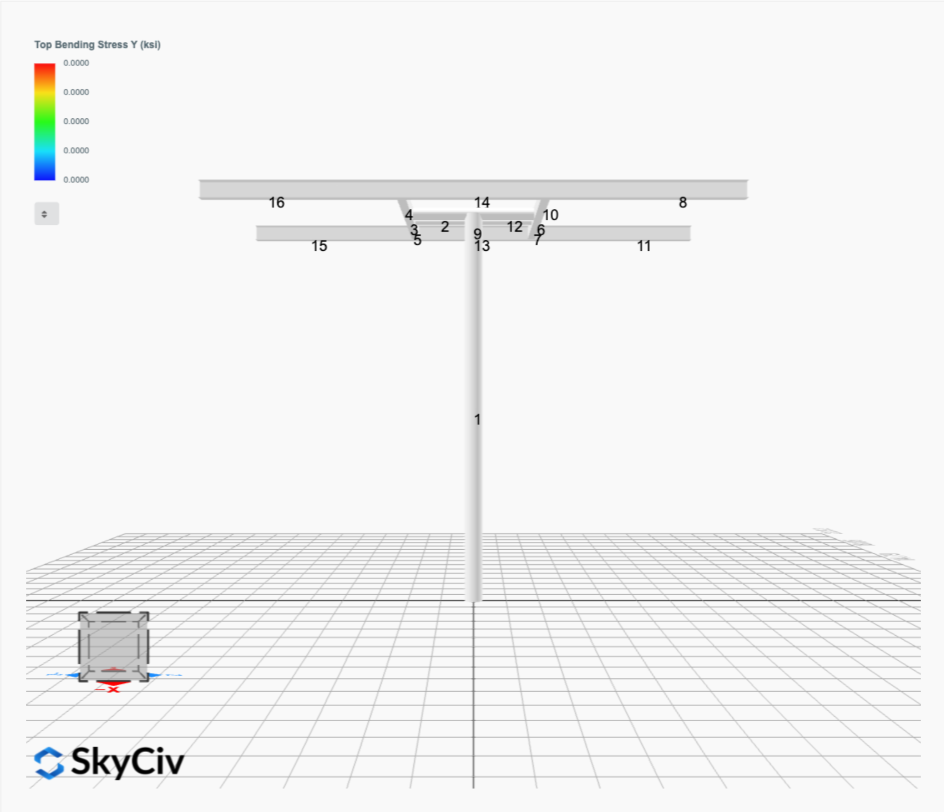


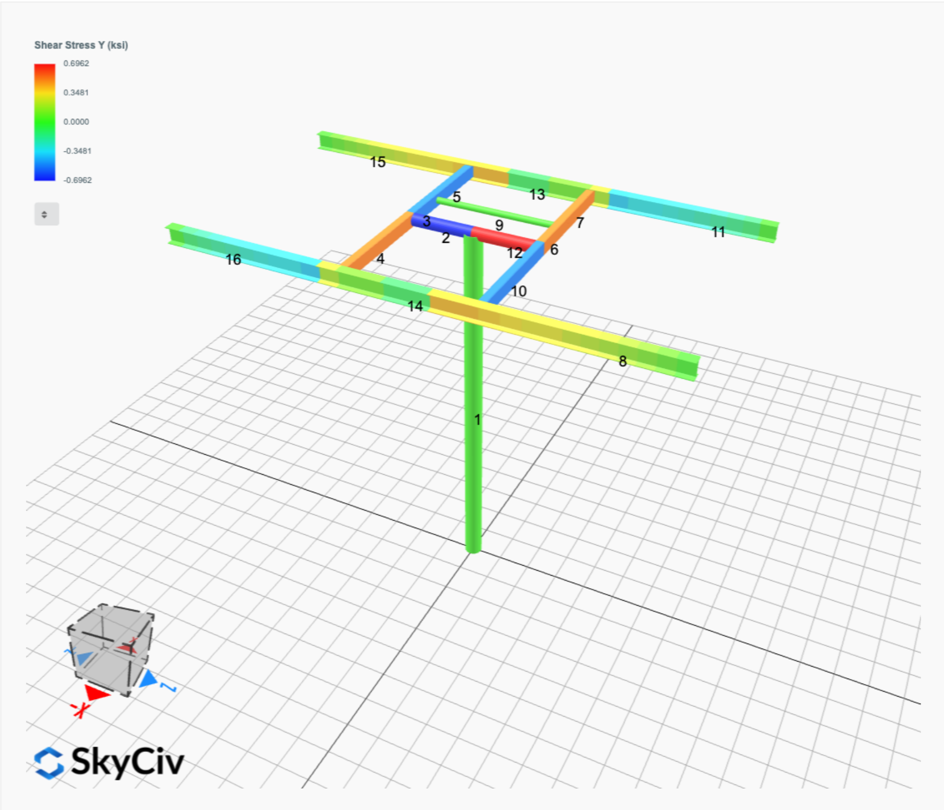


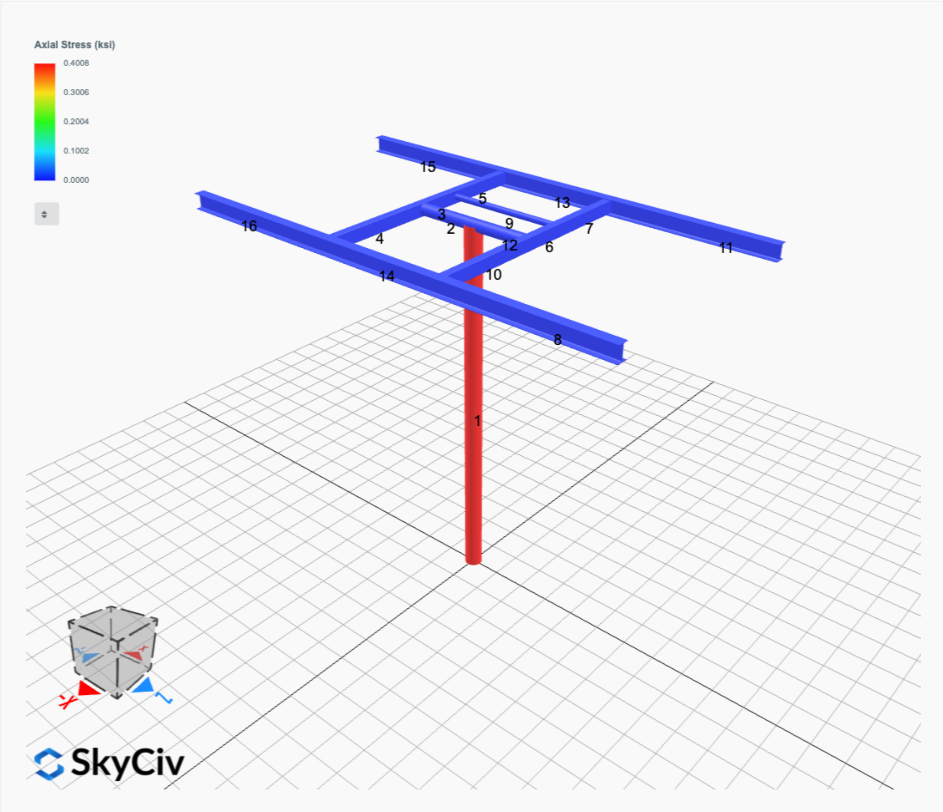
FEM Results (Envelope Worst Case for each member)






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


Shear Stress Y (ksi)
0.6962
0.3481
0.0000
-0.3481
-0.6962
15
5
13
3
9
7
11
2
12
6
16
4
10
14
8
1
SkyCiv


Axial Stress (ksi)
0.4008
0.3006
0.2004
0.1002
0.0000
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5
13
16
3
9
7
4
2
12
6
11
10
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	1.7207	0.0000	0.0000	0.0000	0.0209
ULS: 2. D + L	-0.0000	1.7207	0.0000	0.0000	0.0000	0.0209
ULS: 3. D + (S or Lr or R)	-0.0000	3.9579	0.0000	0.0000	0.0000	0.0209
ULS: 3. D + (S or Lr or R)	-0.0000	1.7207	0.0000	0.0000	0.0000	0.0209
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.3986	0.0000	0.0000	0.0000	0.0209
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.7207	0.0000	0.0000	0.0000	0.0209
ULS: 5b. D + 0.7E	-0.0000	1.7207	0.0000	0.0000	0.0000	0.0209
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.3986	0.0000	0.0000	0.0000	0.0209
ULS: 8. 0.6D + 0.7E	-0.0000	1.0324	0.0000	0.0000	0.0000	0.0125
ULS: 5a. D + 0.6W_Wind downforce Case A only	0.0000	3.2308	0.0000	-0.0000	-0.0000	-3.4885
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	3.2308	0.0000	-0.0000	-0.0000	-3.4885
ULS: 5a. D + 0.6W_Wind uplift Case A only	-0.0000	3.2308	0.0000	-0.0000	-0.0000	3.5303
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	0.5126	0.0000	0.0000	-0.0000	-3.6599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0000	4.5312	0.0000	-0.0000	-0.0000	-2.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.5312	0.0000	-0.0000	-0.0000	-2.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0000	4.5312	0.0000	-0.0000	-0.0000	2.6530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.4925	0.0000	0.0000	-0.0000	-2.7397
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	0.0000	2.8533	0.0000	-0.0000	-0.0000	-2.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.8533	0.0000	-0.0000	-0.0000	-2.6112
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	-0.0000	2.8533	0.0000	-0.0000	-0.0000	2.6530
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	0.8146	0.0000	0.0000	-0.0000	-2.7397
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	0.0000	2.5425	0.0000	-0.0000	-0.0000	-3.4969
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	2.5425	0.0000	-0.0000	-0.0000	-3.4969
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	-0.0000	2.5425	0.0000	-0.0000	-0.0000	3.5220
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	-0.1757	0.0000	0.0000	-0.0000	-3.6682

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	6.9028
Shear X	-0.0000
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	6.3617

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	4.5312
Shear X	-0.0000
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	3.6682

Project Details

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

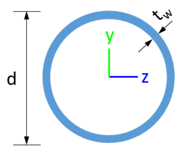
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

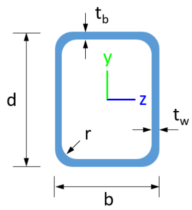
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

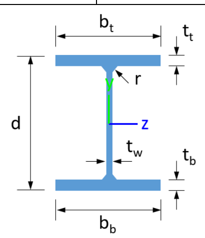
Section Dimensions



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



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



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	25.20	25.20	12.00	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.10,1.18,1.18,1.18,1.17	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.68,1.67,1.67,1.68,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.68,1.67,1.67,1.68,1.69	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.59,1.67,1.67,1.67,1.66	300	200	1
6	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.10,1.18,1.18,1.18,1.17	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.59,1.67,1.67,1.67,1.66	300	200	1
8	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.67,1.68,1.67,1.67,1.68,1.69,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.68,1.67,1.67,1.68,1.69	300	200	1
11	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.22,1.04,1.04,1.04,1.09,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.62,1.04,1.04,1.04,1.08	300	200	1
14	18	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.06,1.04,1.04,1.04,1.13,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.22	300	200	1
15	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1

Member Design Capacity

Member ID	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	251.16	69.52	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	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	54.44	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51

11	120.60	54.44	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	18.31	6.45	30.09	45.74
14	120.60	98.23	18.31	6.45	30.09	45.74
15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	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	0.099	0.150	0.000	0.000	0.000	0.186	#15	0.673	Not Required	Pass
2	0.000	0.373	0.000	0.077	0.000	0.373	#21	Not Required	Not Required	Pass
3	0.000	0.621	0.000	0.063	0.000	0.621	#23	Not Required	Not Required	Pass
4	0.000	0.620	0.000	0.063	0.000	0.620	#21	Not Required	Not Required	Pass
5	0.000	0.384	0.000	0.062	0.000	0.384	#23	Not Required	Not Required	Pass
6	0.000	0.621	0.000	0.063	0.000	0.621	#23	Not Required	Not Required	Pass
7	0.000	0.384	0.000	0.062	0.000	0.384	#23	Not Required	Not Required	Pass
8	0.000	0.072	0.000	0.030	0.000	0.072	#21	Not Required	Not Required	Pass
9	0.000	0.067	0.000	0.001	0.000	0.067	#23	Not Required	Not Required	Pass
10	0.000	0.620	0.000	0.063	0.000	0.620	#21	Not Required	Not Required	Pass
11	0.000	0.072	0.000	0.030	0.000	0.072	#23	Not Required	Not Required	Pass
12	0.000	0.373	0.000	0.077	0.000	0.373	#21	Not Required	Not Required	Pass
13	0.000	0.206	0.000	0.044	0.000	0.206	#23	Not Required	Not Required	Pass
14	0.000	0.210	0.000	0.044	0.000	0.210	#21	Not Required	Not Required	Pass
15	0.000	0.072	0.000	0.030	0.000	0.072	#23	Not Required	Not Required	Pass
16	0.000	0.072	0.000	0.030	0.000	0.072	#21	Not Required	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS

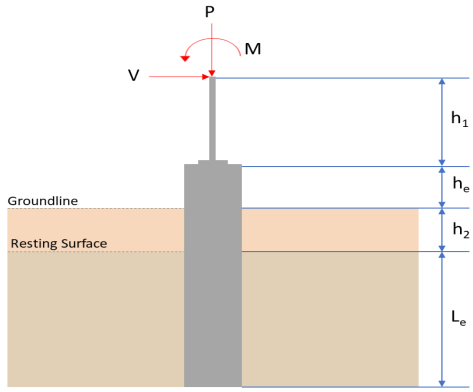
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 3.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.531	6.903
V_x (kip)	0.000	0.000
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	3.668	6.362

Material Properties

$f'_{ck} = 3\ \text{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0\ \text{kip})}{1.57 \times (48\ \text{in})}$$

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(3.668\ \text{kipft}) + ((0\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.58408\ \text{kipft/ft}$$

L_e - Required depth of embedment in earth,

$$L_e = 2.29 \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.58408\ \text{kipft/ft})}{(150\ \text{psf/ft})}}$$

$$L_e = 3.6027\ \text{ft}$$

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,

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

$$L_{e,req} = MAX[(3.6027\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 3.603\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (3.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 3.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(3.603\ \text{ft})}{(3.75\ \text{ft})}$$

$$Ratio = 0.9608$$

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

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

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

$$q = \frac{(4.531\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.28319\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.28319\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.14159$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(3.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 0.9375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = 0\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.58408\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.58408\ \mathrm{kipft/ft}) \times (3.75\ \mathrm{ft})) + \left(3 \times (0\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft})^2\right)}{(6 \times (0.58408\ \mathrm{kipft/ft})) + (4 \times (0\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))}$$

$$a = 2.5\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.58408\ \mathrm{kipft/ft})) + (3 \times (0\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]^{\ 2}}{(3.75\ \mathrm{ft})^2 \times [(3 \times (0.58408\ \mathrm{kipft/ft})) + (2 \times (0\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]}$$

$$p = 0.16614\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.58408\ \mathrm{kipft/ft})) + ((0\ \mathrm{kip/ft}) \times (3.75\ \mathrm{ft}))]}{(3.75\ \mathrm{ft})^2}$$

$$s = 0.49841\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(2.5\ \mathrm{ft})}{2}$$

$$p_a = 0.1875\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.16614\ \mathrm{kip/ft^2}\right)}{\left(0.1875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88607$$

Status: **PASS**
Ratio: **0.890**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\ L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (3.75\ \mathrm{ft})$$

$$p_s = 0.5625\ \mathrm{kip/ft^2}$$

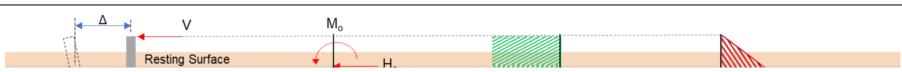
$Ratio$ - Lateral soil capacity

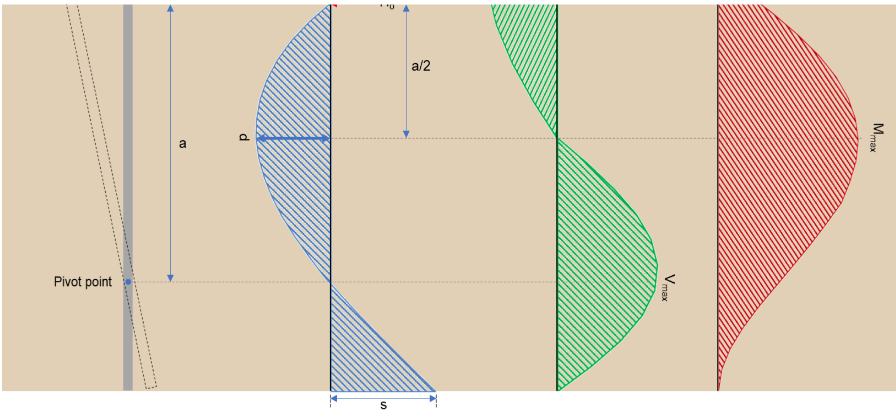
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.49841\ \mathrm{kip/ft^2}\right)}{\left(0.5625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.88607$$

Status: **PASS**
Ratio: **0.890**





Pile Rotation | **Pressure Diagram** | **Shear Diagram** | **Moment Diagram**

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0\ \text{kip})}{1.57 \times (48\ \text{in})}$$

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(6.362\ \text{kipft}) + ((0\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 1.0131\ \text{kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.0131\ \text{kipft/ft}) \times (3.75\ \text{ft})) + \left(3 \times (0\ \text{kip/ft}) \times (3.75\ \text{ft})^2\right)}{(6 \times (1.0131\ \text{kipft/ft})) + (4 \times (0\ \text{kip/ft}) \times (3.75\ \text{ft}))}$$

$$a = 2.5\ \text{ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(1.0131\ \text{kipft/ft}) \times (48\ \text{in})}{(3.75\ \text{ft})}\right) \times \left(\frac{(2.5\ \text{ft})}{(3.75\ \text{ft})} - 1\right) \times \left(\frac{(2.5\ \text{ft})}{(3.75\ \text{ft})}\right)^2$$

$$V_{max} = 1.9211\ \text{kip}$$

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

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

$$M_{max} = ((1.0131\ \text{kipft/ft}) \times (48\ \text{in})) \times \left[1 - \left(4 \times \frac{(2.5\ \text{ft})}{2 \times (3.75\ \text{ft})}^3\right) + \left(3 \times \frac{(2.5\ \text{ft})}{2 \times (3.75\ \text{ft})}^4\right)\right]$$

$$M_{max} = 3.602\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3\ \text{ksi}$ - 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(6.903\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (3\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (3\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -102.04\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-102.04\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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

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

$$\phi P_N = (0.65) \times 0.80 \times \left[(0.85 \times (3\ \text{ksi}) \times [(2304\ \text{in}^2) - (4.2951\ \text{in}^2)]) + ((60\ \text{ksi}) \times (4.2951\ \text{in}^2))\right]$$

$$\phi P_N = 3183.4\ \text{kip}$$

$Ratio$ - Capacity

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

	$$Ratio = \frac{(6.903\ \mathrm{kip})}{(3183.4\ \mathrm{kip})}$$ $$Ratio = 0.0021684$$		Status: **PASS** Ratio: **0.000**

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,max} = 324.49\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $P = 6.903\ \mathrm{kip} \rightarrow 6903\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + \frac{(6903\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 130.71\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + (0.05 \times (3000\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,b} = 406.27\ \mathrm{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49\ \mathrm{kip}), (130.71\ \mathrm{kip}), (406.27\ \mathrm{kip})\right]$$

$$V_c = 130.71\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$,

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

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

$$V_{s,a} = 8 \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{s,a} = 807.65\ \mathrm{kip}$$

A_v - Ties rebar area,

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

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

$$A_v = \frac{\quad}{4}$$

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

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 $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(807.65 \text{ kip}), (50.894 \text{ kip})\right]$$ $$V_s = 50.894 \text{ kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((130.71 \text{ kip}) + (50.894 \text{ kip}))$$ $$\phi V_n = 118.05 \text{ kip}$$	
	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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\ 0.85\ f_{ck}'\ S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2545.9 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$ $$\phi M_n = 273.42 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 3.602$ kipft - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(3.602 \text{ kipft})}{(273.42 \text{ kipft})}$$ $$Ratio = 0.013174$$	Status: **PASS** Ratio: **0.010**