Your Project Calculations

Project Name: Conlee-RevA

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

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=MZhgqZYVXcfUNN0EJN322sf1UBK6ME4T22cSlg0Q2oMFpuefZ0NDDrk6q1pgitAY

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-HD-24-L-4Hx2W-20I2
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	4
Number of Columns:	2
Total Number of Modules:	8
Desired Tilt Angle:	35
Front Edge Clearance:	5
Total Array Height at Tilt:	12.91 ft
Total Frame Length:	11.50 ft
Frame Weight:	614 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	12.50 ft
Rail Length:	166.40 in
Rail Spacing:	3.08 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.98 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: 5.00 ft
Foundation Volume:	2.963 y^3
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1840 Aspen Dr, Ridgway, CO 81432, USA
Wind Speed:	98 mph
Snow Load:	120 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.037832 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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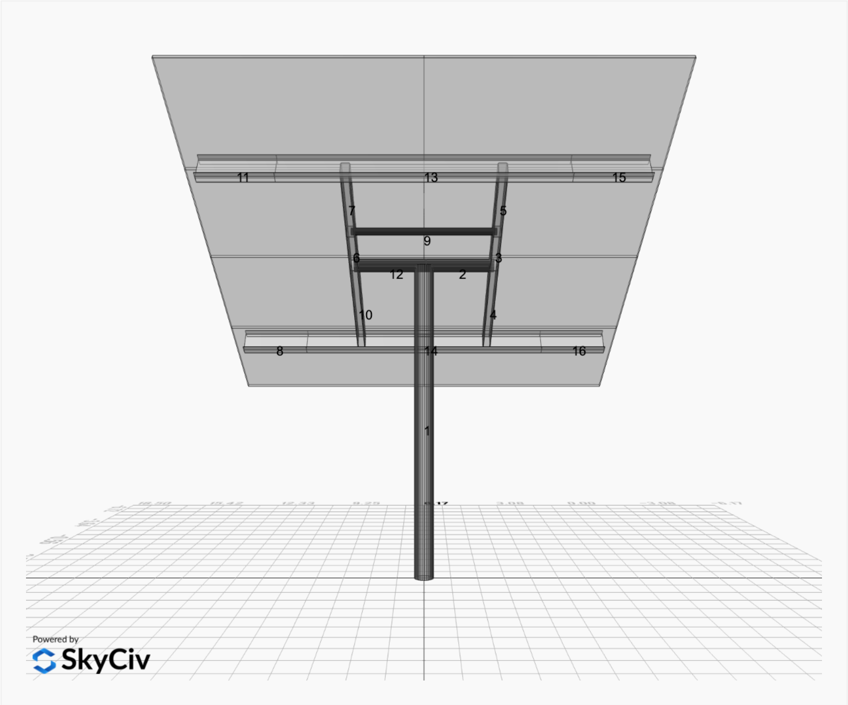
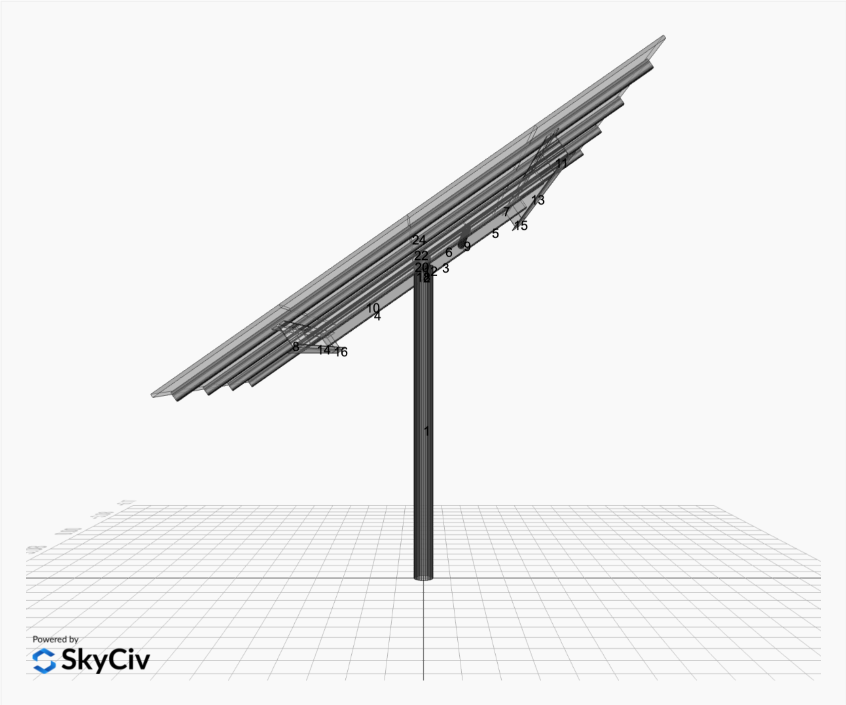
Design Notes:

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

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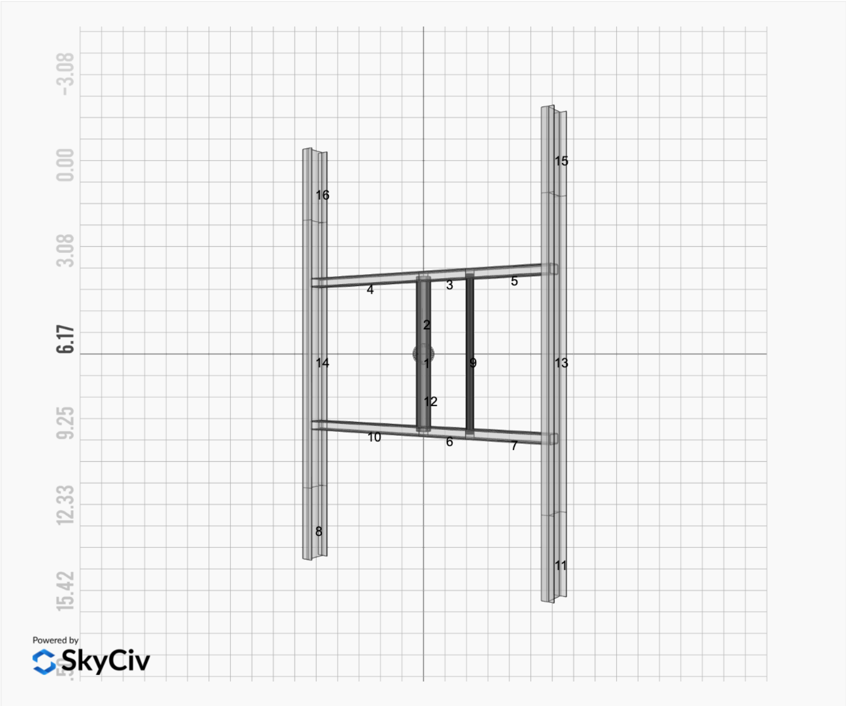
SkyCiv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3.08
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12.33
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13
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10
6
7
8
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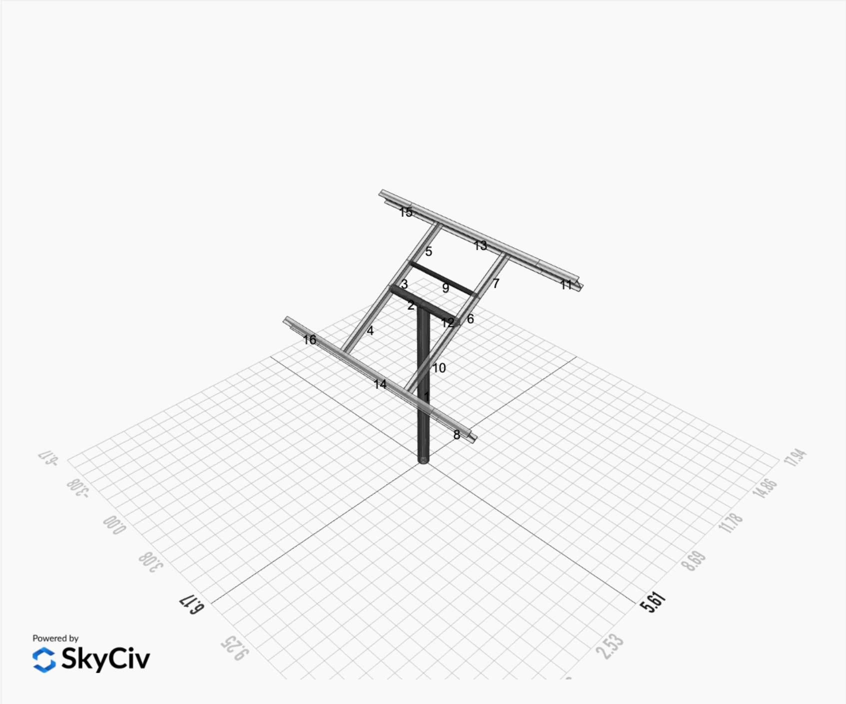
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8.69
5.61
2.53
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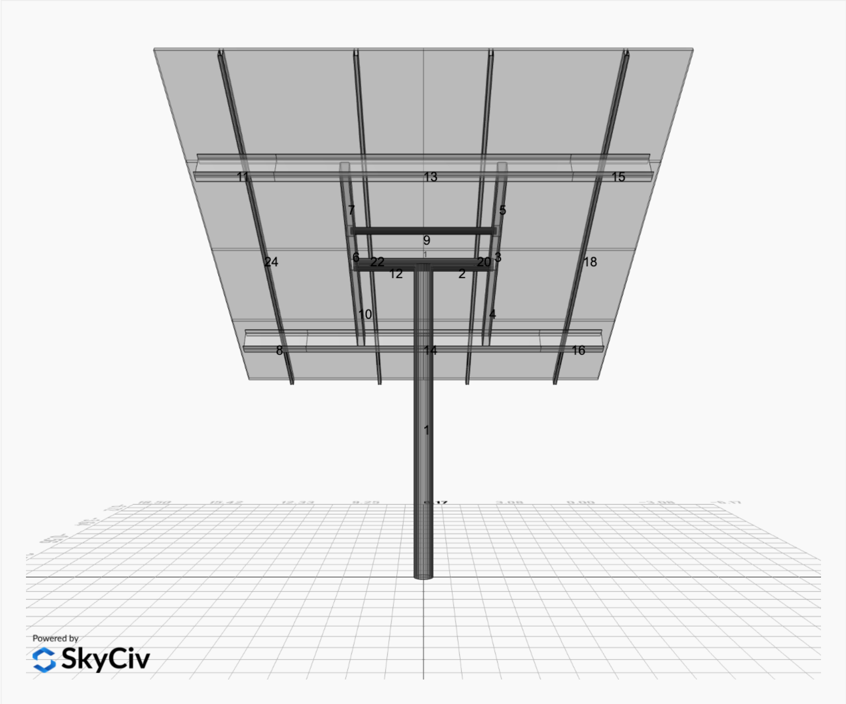
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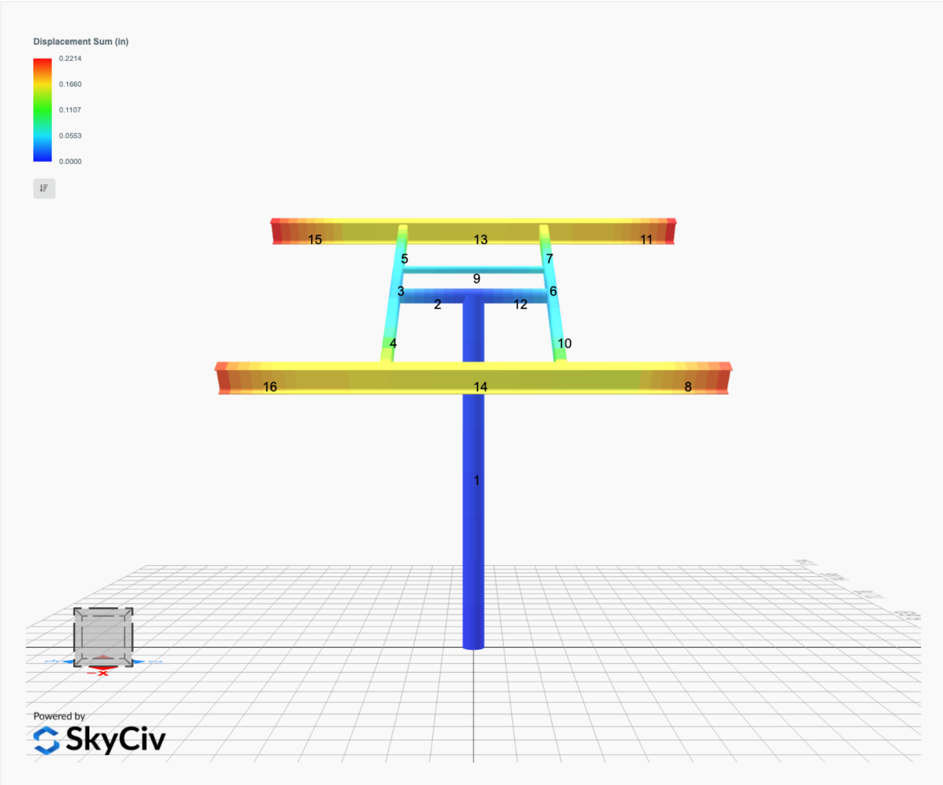
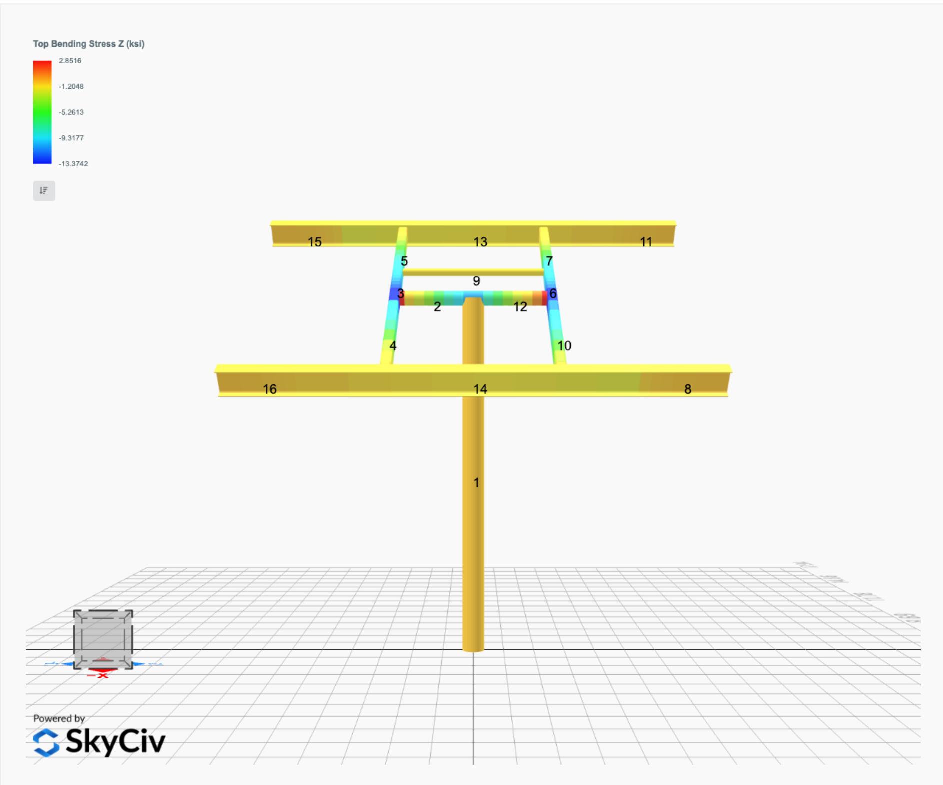
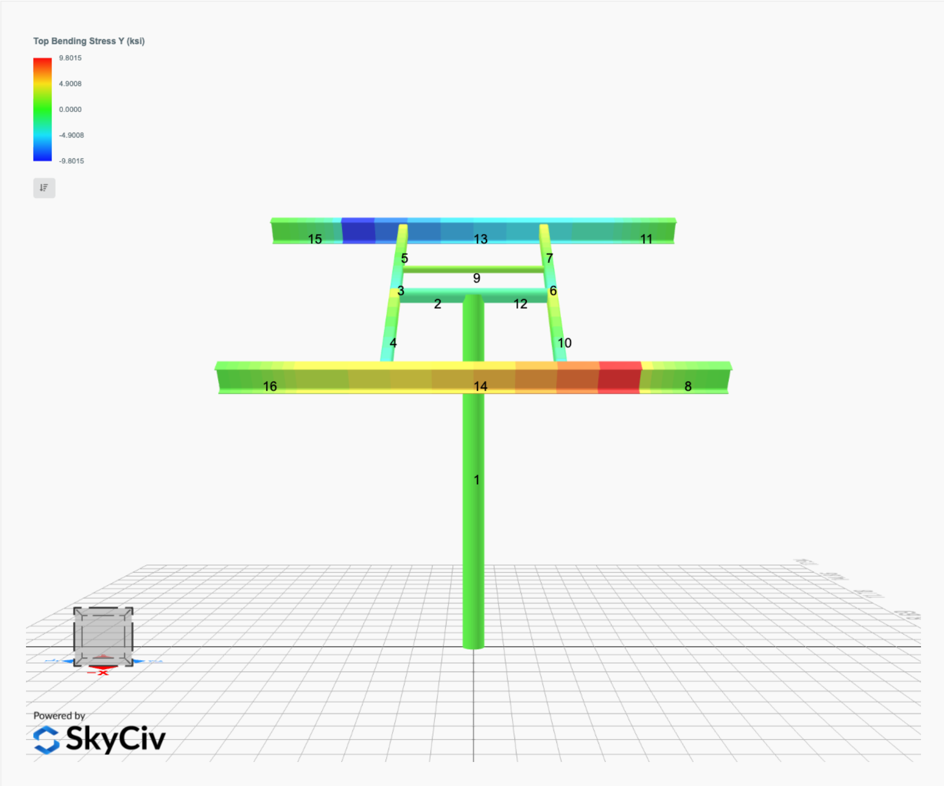
Powered by SkyCiv

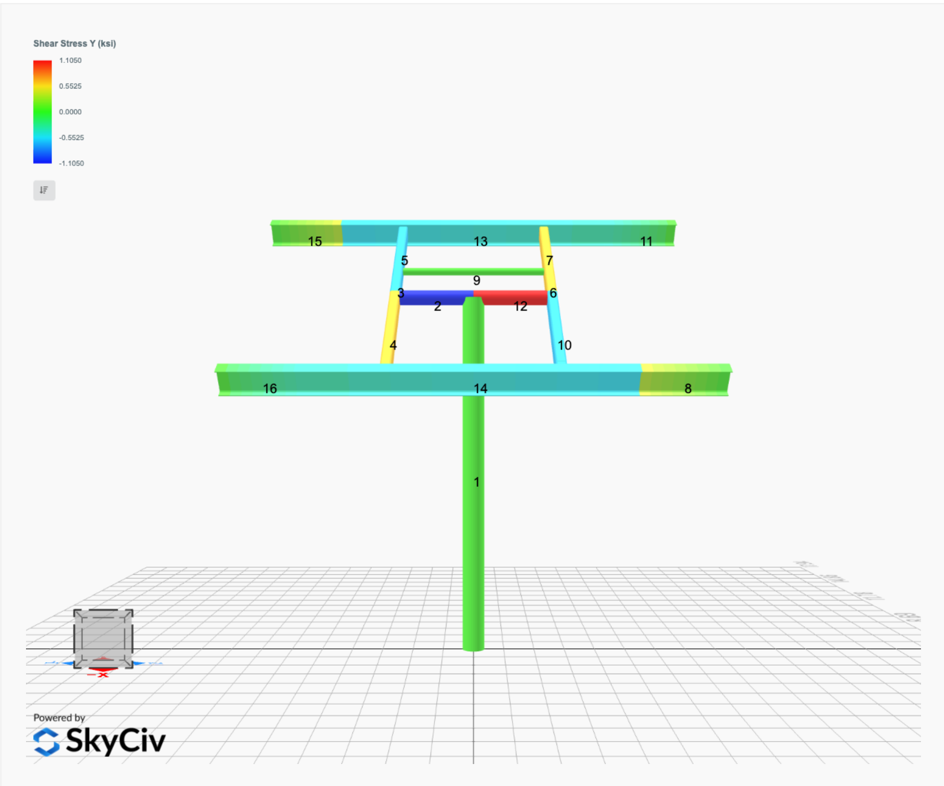
FEM Results (Envelope Worst Case for each member)



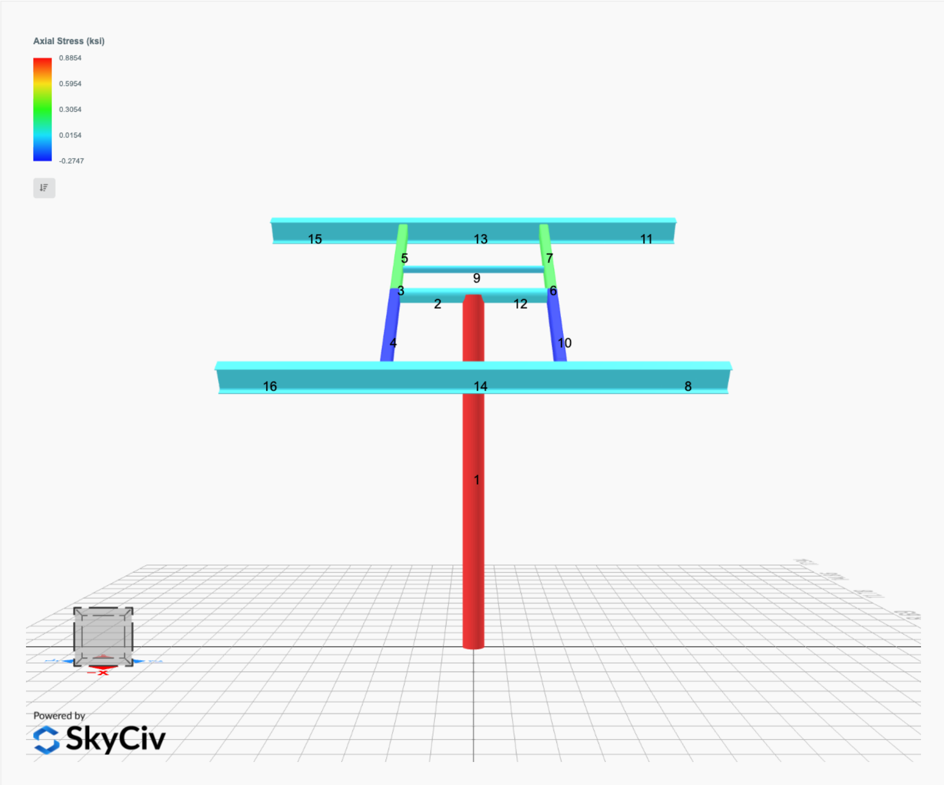




Top Bending Stress Y (ksi)
9.8015
4.9008
0.0000
-4.9008
-9.8015
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
Powered by
SkyCiv



Shear Stress Y (ksi)
1.1050
0.5525
0.0000
-0.5525
-1.1050
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
Powered by
SkyCiv


Axial Stress (ksi)
0.8854
0.5954
0.3054
0.0154
-0.2747
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
Powered by
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	1.4273	0.0000	0.0000	-0.0000	0.0243
ULS: 2. D + L	0.0000	1.4273	0.0000	0.0000	-0.0000	0.0243
ULS: 3. D + (S or Lr or R)	0.0000	6.3692	-0.0000	0.0000	-0.0000	0.0548
ULS: 3. D + (S or Lr or R)	0.0000	1.4273	0.0000	0.0000	-0.0000	0.0243
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	5.1337	-0.0000	0.0000	-0.0000	0.0472
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.4273	0.0000	0.0000	-0.0000	0.0243
ULS: 5b. D + 0.7E	0.0000	1.4273	0.0000	0.0000	-0.0000	0.0243
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	5.1337	-0.0000	0.0000	-0.0000	0.0472
ULS: 8. 0.6D + 0.7E	0.0000	0.8564	0.0000	0.0000	-0.0000	0.0146
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4006	3.4276	0.0000	0.0000	-0.0000	12.9465
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4006	3.4276	0.0000	0.0000	-0.0000	12.9465
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1818	-0.2605	-0.0000	0.0000	-0.0000	-10.4451
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9848	0.0208	0.0000	0.0000	-0.0000	-12.6597
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0505	6.6340	0.0000	0.0000	-0.0000	9.7388
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0505	6.6340	0.0000	0.0000	-0.0000	9.7388
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8863	3.8679	-0.0000	0.0000	-0.0000	-7.8049
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7386	4.0789	-0.0000	0.0000	-0.0000	-9.4659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0505	2.9275	0.0000	0.0000	-0.0000	9.7160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0505	2.9275	0.0000	0.0000	-0.0000	9.7160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8863	0.1614	-0.0000	0.0000	-0.0000	-7.8277
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7386	0.3724	0.0000	0.0000	-0.0000	-9.4887
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4006	2.8567	0.0000	0.0000	-0.0000	12.9368
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4006	2.8567	0.0000	0.0000	-0.0000	12.9368
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1818	-0.8314	-0.0000	0.0000	-0.0000	-10.4548
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9848	-0.5501	0.0000	0.0000	-0.0000	-12.6694

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
Axial	11.2867
Shear X	-2.3344
Shear Z	0.0000
Moment X	-0.0000
Moment Z	22.0595

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
Axial	6.6340
Shear X	-1.4006
Shear Z	0.0000
Moment X	0.0000
Moment Z	12.9465

Project Details

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

Design Input Information

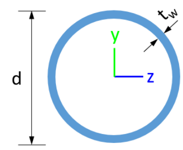
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

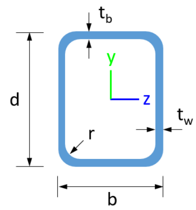
Design Materials

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

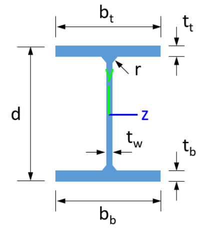
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
7	6in Pipe Sch 40	6.63	0.28					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

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)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	7	18.85	18.85	8.98	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.13,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.68,1.70,1.67,1.67,1.58,1.68,1.67,1.67,1.65,1.11,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.73,1.67,1.67,1.66,1.59	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.62,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.13,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.17,1.17	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.62,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
8	19	4.20	4.20	2.00	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	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.68,1.70,1.67,1.67,1.58,1.68,1.67,1.67,1.65,1.11,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.73,1.67,1.67,1.66,1.59	300	200	1
11	19	4.20	4.20	2.00	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	5	1.30	1.30	2.00	-	300	200	1
13	19	1.14	1.14	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
14	19	1.14	1.14	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
15	19	4.20	4.20	2.00	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
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18	19	1.14	1.14	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
19	19	2.60	2.60	4.00	1.19,1.19,1.19,1.20,1.19,1.19,1.20,1.20,1.22,1.24,1.20,1.20,1.20,1.03,1.20,1.20,1.20,1.20,1.19,1.19,1.21,1.30,1.20,1.20,1.20,1.09	300	200	1
20	19	1.14	1.14	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	119.58	42.30	42.30	75.35	75.35

2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	102.39	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	102.39	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	126.79	32.87	6.12	40.24	43.62
14	133.20	126.79	32.87	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62
17	133.20	118.19	32.87	6.12	40.24	43.62
18	133.20	126.79	32.87	6.12	40.24	43.62
19	133.20	118.19	31.83	6.12	40.24	43.62
20	133.20	126.79	32.87	6.12	40.24	43.62

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	0.094	0.522	0.000	0.031	0.000	0.553	#13	0.504	Not Required	Pass
2	0.001	0.406	0.115	0.093	0.020	0.504	#21	0.053	Not Required	Pass
3	0.012	0.578	0.095	0.058	0.028	0.679	#21	0.045	Not Required	Pass
4	0.012	0.569	0.099	0.057	0.024	0.663	#21	0.080	Not Required	Pass
5	0.012	0.359	0.098	0.057	0.023	0.378	#21	0.074	Not Required	Pass
6	0.012	0.578	0.095	0.058	0.028	0.679	#21	0.045	Not Required	Pass
7	0.012	0.359	0.098	0.057	0.023	0.378	#21	0.074	Not Required	Pass
8	0.000	0.025	0.075	0.021	0.010	0.100	#21	Not Required	Not Required	Pass
9	0.002	0.042	0.039	0.001	0.000	0.083	#21	0.136	Not Required	Pass
10	0.012	0.569	0.099	0.057	0.024	0.663	#21	0.080	Not Required	Pass
11	0.000	0.025	0.075	0.021	0.010	0.100	#21	Not Required	Not Required	Pass
12	0.001	0.406	0.115	0.093	0.020	0.504	#21	0.053	Not Required	Pass
13	0.000	0.089	0.263	0.039	0.020	0.352	#21	Not Required	Not Required	Pass
14	0.000	0.088	0.263	0.039	0.020	0.351	#21	Not Required	Not Required	Pass
15	0.000	0.025	0.075	0.021	0.010	0.100	#21	Not Required	Not Required	Pass
16	0.000	0.025	0.075	0.021	0.010	0.100	#21	Not Required	Not Required	Pass
17	0.004	0.103	0.085	0.021	0.010	0.189	#21	0.124	Not Required	Pass
18	0.000	0.089	0.263	0.039	0.020	0.352	#21	Not Required	Not Required	Pass
19	0.005	0.104	0.084	0.021	0.010	0.190	#21	0.186	Not Required	Pass
20	0.000	0.088	0.263	0.039	0.020	0.351	#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

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

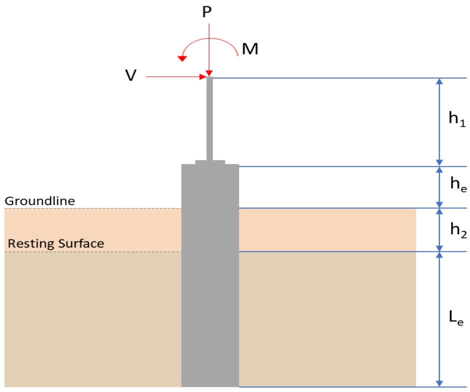
SkyCiv Foundation Design

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
$L = 5\ \mathrm{ft}$ - 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)	6.634	11.287
V_x (kip)	-1.401	-2.334
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	12.947	22.059

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - 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,

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

$$H_o = \frac{(-1.401\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.22309\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(12.947\ \text{kipft}) + ((-1.401\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 2.0616\ \text{kipft/ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,x} = 4.6773\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

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

$$L_{e,req} = MAX[(4.6773\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 4.677\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 5\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.677\ \text{ft})}{(5\ \text{ft})}$$

$$Ratio = 0.9354$$

Status: **PASS**
Ratio: **0.940**

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

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

$$q = \frac{(6.634\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.41463\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.41463\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.20731$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.25$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.22309 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 2.0616 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0616 \text{ kipft/ft}) \times (5 \text{ ft})) + \left(3 \times (-0.22309 \text{ kip/ft}) \times (5 \text{ ft})^2\right)}{(6 \times (2.0616 \text{ kipft/ft})) + (4 \times (-0.22309 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4438 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.0616 \text{ kipft/ft})) + (3 \times (-0.22309 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (2.0616 \text{ kipft/ft})) + (2 \times (-0.22309 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.18218 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.0616 \text{ kipft/ft})) + ((-0.22309 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.72187 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4438 \text{ ft})}{2}$$

$$p_a = 0.25828 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.18218 \text{ kip/ft}^2)}{(0.25828 \text{ kip/ft}^2)}$$

$$Ratio = 0.70536$$

Status: **PASS**
Ratio: **0.710**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

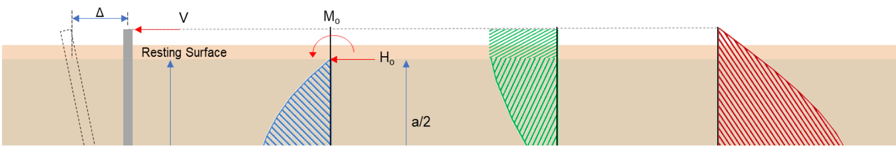
$Ratio$ - Lateral soil capacity

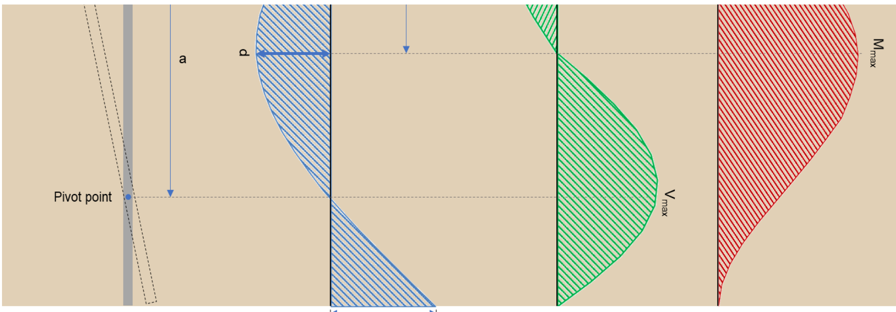
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.72187 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.9625$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.334\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.37166\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(22.059\text{ kipft}) + ((-2.334\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 3.5126\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.5126\text{ kipft/ft})}{(-0.37166\text{ kip/ft})}$$

$$E = 9.4512\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.5126\text{ kipft/ft}) \times (5\text{ ft})) + \left(3 \times (-0.37166\text{ kip/ft}) \times (5\text{ ft})^2\right)}{(6 \times (3.5126\text{ kipft/ft})) + (4 \times (-0.37166\text{ kip/ft}) \times (5\text{ ft}))}$$

$$a = 3.442\text{ ft}$$

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

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

$$V_{max} = ((-0.37166\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.4512\text{ ft})}{(5\text{ ft})} + 3\right) \times \left(\frac{(3.442\text{ ft})}{(5\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.4512\text{ ft})}{(5\text{ ft})} + 2\right) \times \left(\frac{(3.442\text{ ft})}{(5\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 5.9535\text{ kip}$$

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

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

$$M_{max} = ((-0.37166\text{ kip/ft}) \times (48\text{ in}) \times (5\text{ ft})) \times \left[\left(\frac{(9.4512\text{ ft})}{(5\text{ ft})} + \frac{(3.442\text{ ft})}{2 \times (5\text{ ft})}\right) - \left[\left(\frac{4 \times (9.4512\text{ ft})}{(5\text{ ft})} + 3\right) \times \left(\frac{(3.442\text{ ft})}{2 \times (5\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (9.4512\text{ ft})}{(5\text{ ft})} + 2\right) \times \left(\frac{(3.442\text{ ft})}{2 \times (5\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 14.208 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.287 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -101.89 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-101.89 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.287\ \text{kip})}{(3183.4\ \text{kip})}$$

$$Ratio = 0.0035456$$

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 $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$,

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \text{ksi} \rightarrow 3000\ \text{psi}$, $P = 11.287\ \text{kip} \rightarrow 11287\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \text{psi})} + \frac{(11287\ \text{lbf})}{6 \times \left(2304\ \text{in}^2\right)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 131.3\ \text{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(324.49\ \text{kip}), (131.3\ \text{kip}), (406.27\ \text{kip})\right]$$

$$V_c = 131.3\ \text{kip}$$

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $V_{s,a}$ - 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, $$A_v = \frac{\pi\,d_{ties}^2}{4}$$ $$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$ $$A_v = 0.11045\ \mathrm{in}^2$$	
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 $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(807.65\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$ $$V_s = 50.894\ \mathrm{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((131.3\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 118.43\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 5.9535\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(5.9535\ \mathrm{kip})}{(118.43\ \mathrm{kip})}$$ $$Ratio = 0.050272$$	Status: **PASS** Ratio: **0.050**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 273.423 kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\,0.85\,f'_c\,S_m$$	

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

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

$$\phi M_{n,2} = 2545.9\ \text{kipft}$$

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:

$M_{max} = 14.208\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(14.208\ \text{kipft})}{(273.42\ \text{kipft})}$$

$$Ratio = 0.051963$$

Status: **PASS**
Ratio: **0.050**