Your Project Calculations

Project Name: 1211ArrowheadDriveSolar-RevD

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=1211ArrowheadDriveSolar-RevD&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/3_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=4Wa7Zkfdk8Z5LE0xwfTVhQ0fJ7ySXbuNjvlfG6TbScKIBpbqaIMPm48wc85PH078

Array Specification

Product:	Beam
Unique ID:	1P-0-10TOP-XD-72-L-4Hx3W-D32L
Duty Classification:	XD
Module Width:	39.00 in
Module Length:	79.00in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	45
Front Edge Clearance:	5
Total Array Height at Tilt:	14.25 ft
Total Frame Length:	19.50 ft
Frame Weight:	1132 lbs
Array Dimensions N/S:	13.17 ft
Array Dimensions E/W:	20.00 ft
Rail Length:	158.00 in
Rail Spacing:	3.29 ft
Rail Check:	

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	9.66 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 15.00 ft
Foundation Volume:	3.927 y^3
Foundation Result:	FAILED Try increasing foundation size and/or type and re-running foundation design check on right panel.

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	1211 Arrowhead Dr, Jefferson, CO 80456, USA
Wind Speed:	140 mph
Snow Load:	50 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.009720 ksf

Design Disclaimer

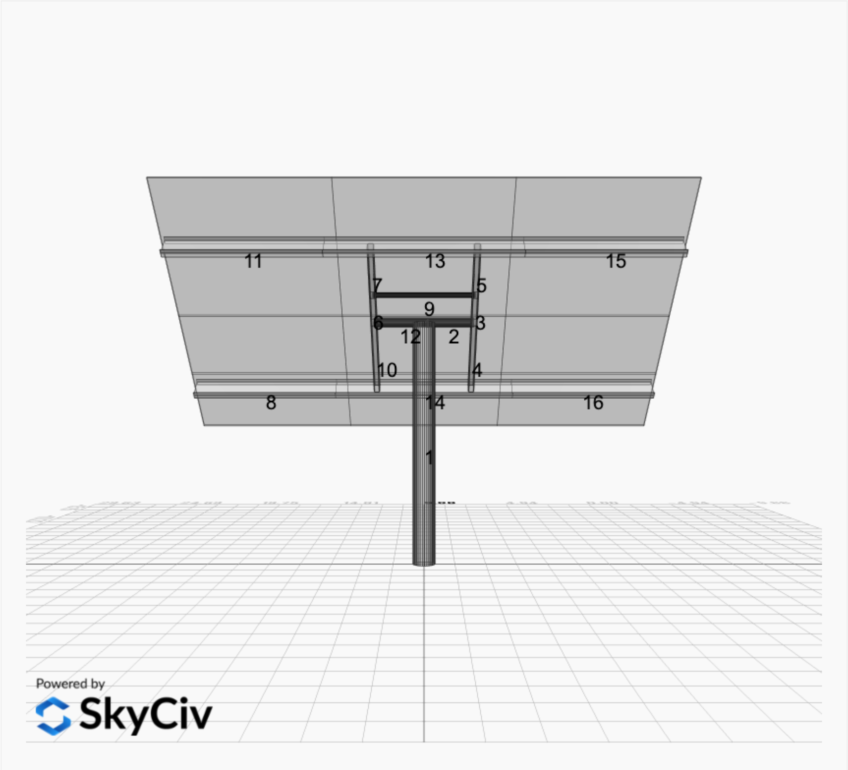
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

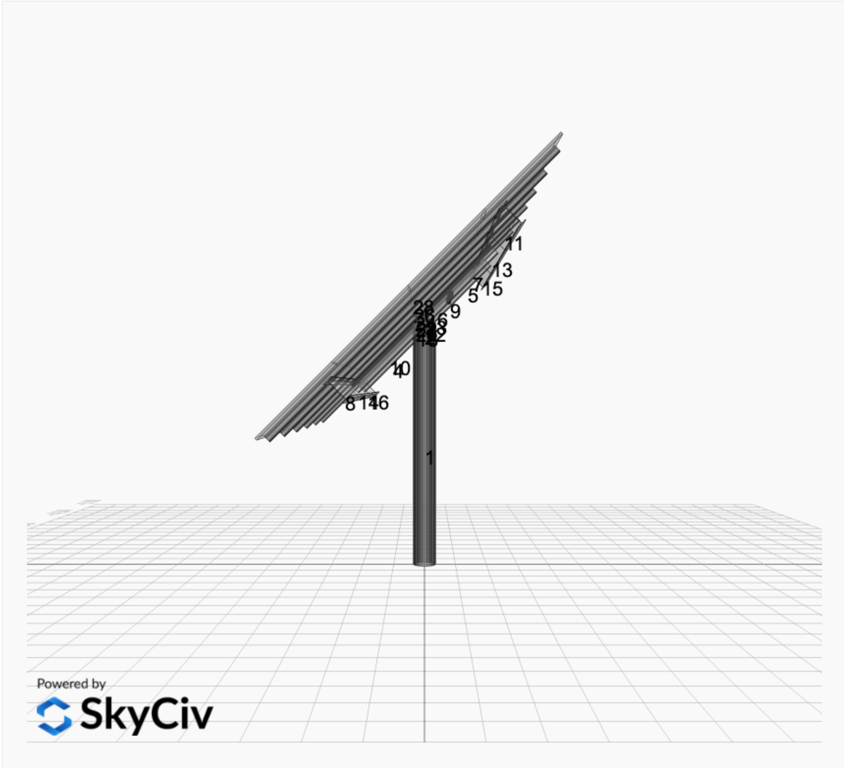
AutoDesigner Input

{"wind_speed_override":140,"snow_load_override":50,"direct_snow_load":false,"product_type":"Beam","project_id":"1211ArrowheadDriveSolar-RevD","site_address":"1211 Arrowhead Dr, Jefferson, CO 80456, USA","module_width":39,"module_length":79,"number_rows":"4","number_columns":3,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":45,"ground_clearance":5,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","foundation_type":"Round","foundation_size":36,"check_rails":true}

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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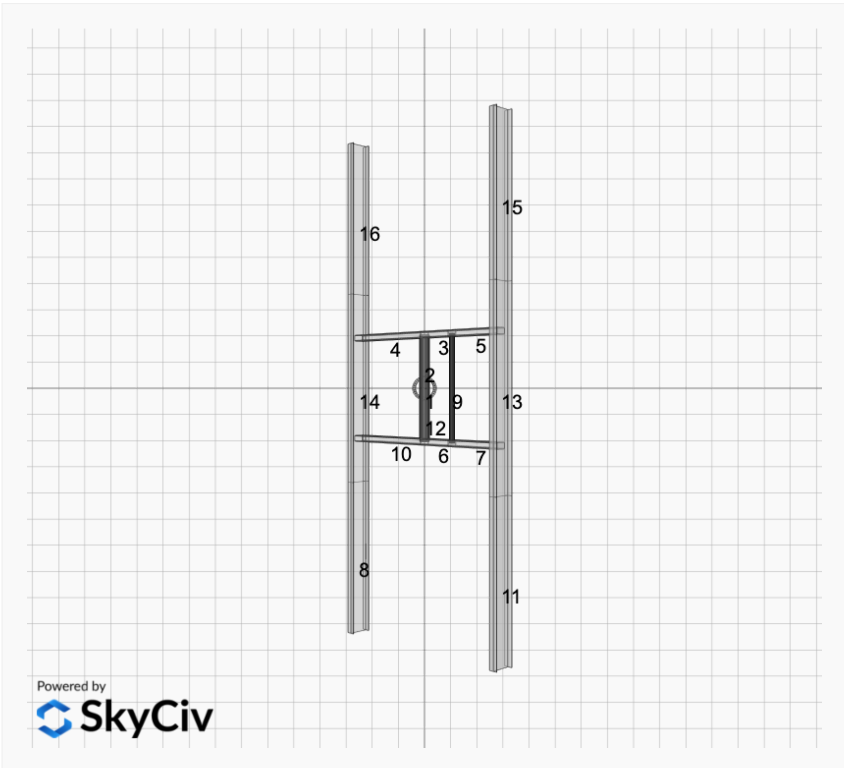
SkyCiv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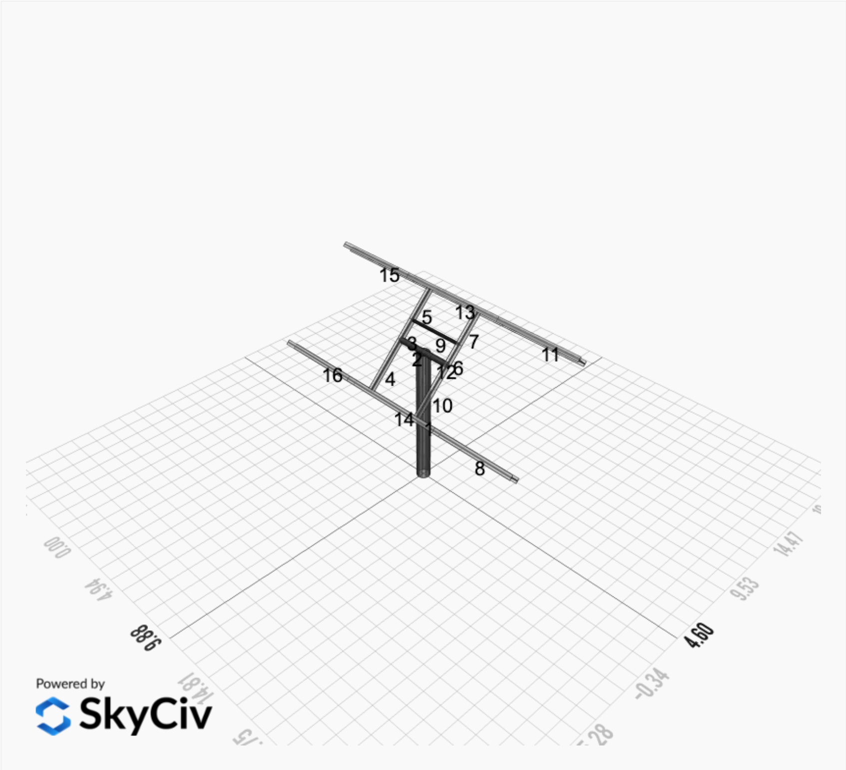
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4.60
9.53
14.47
-0.34
Powered by
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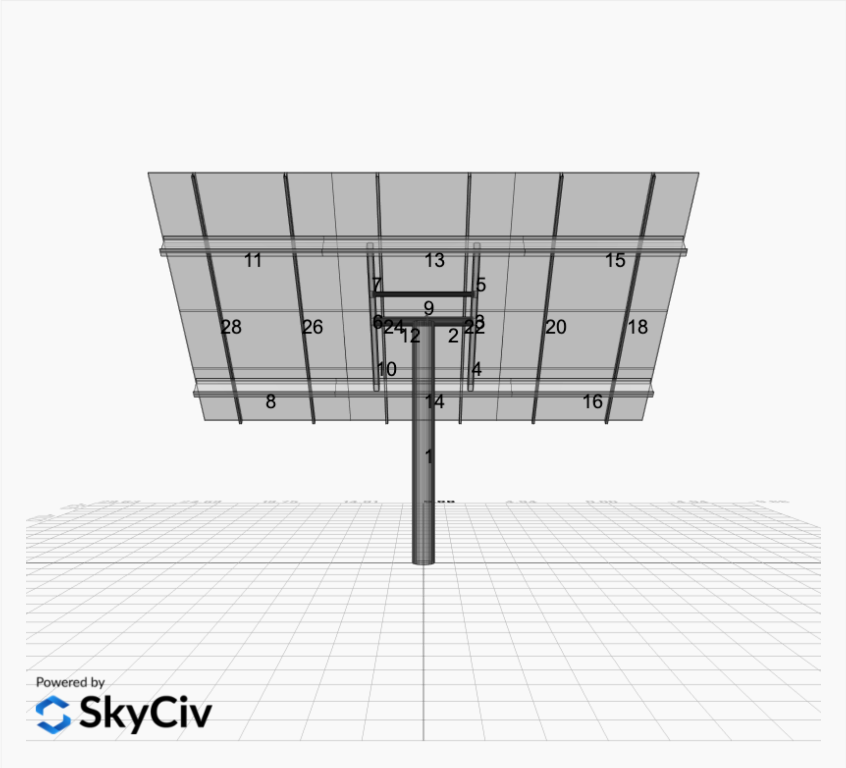
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5
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18
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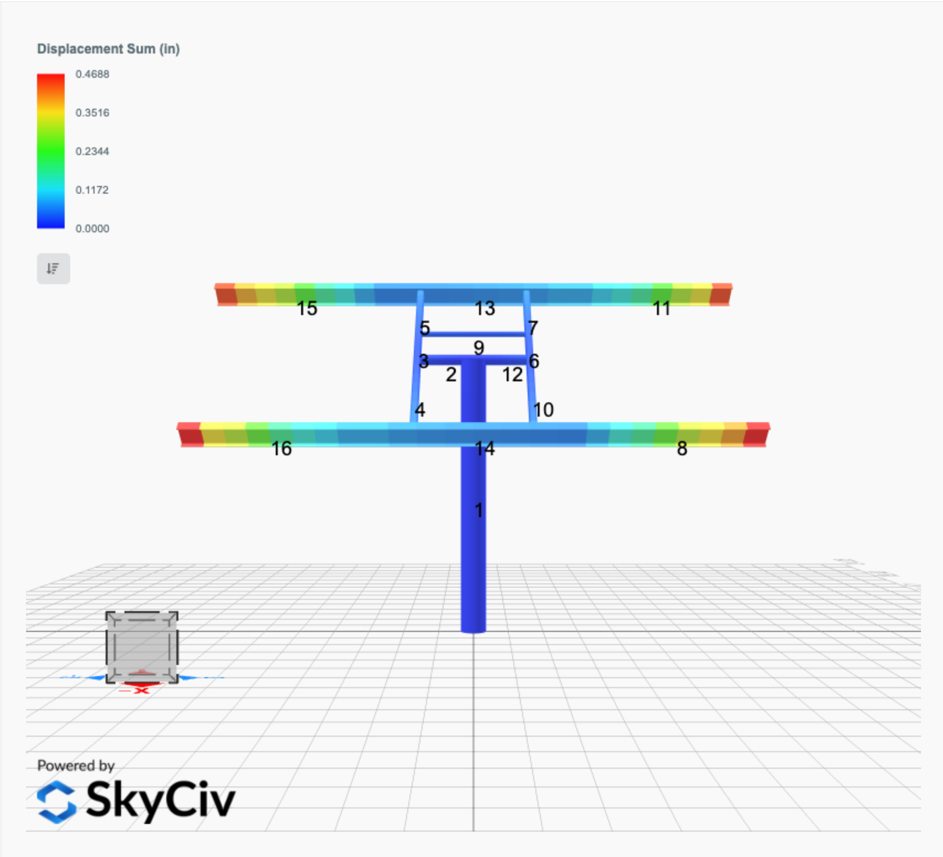
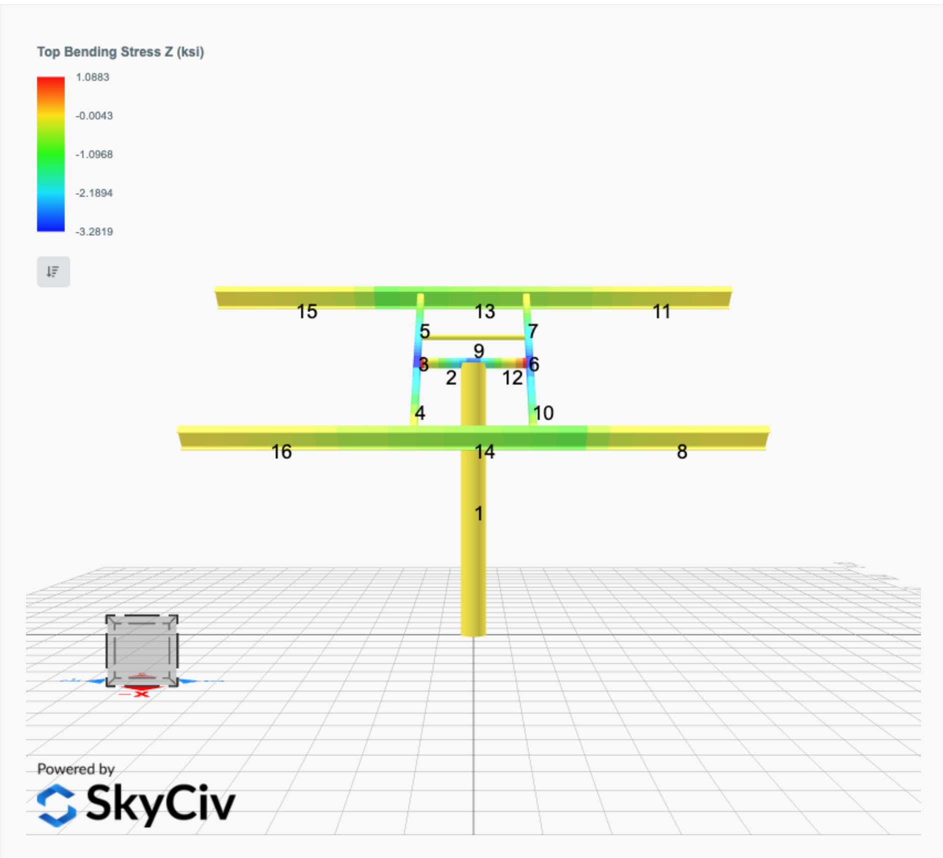
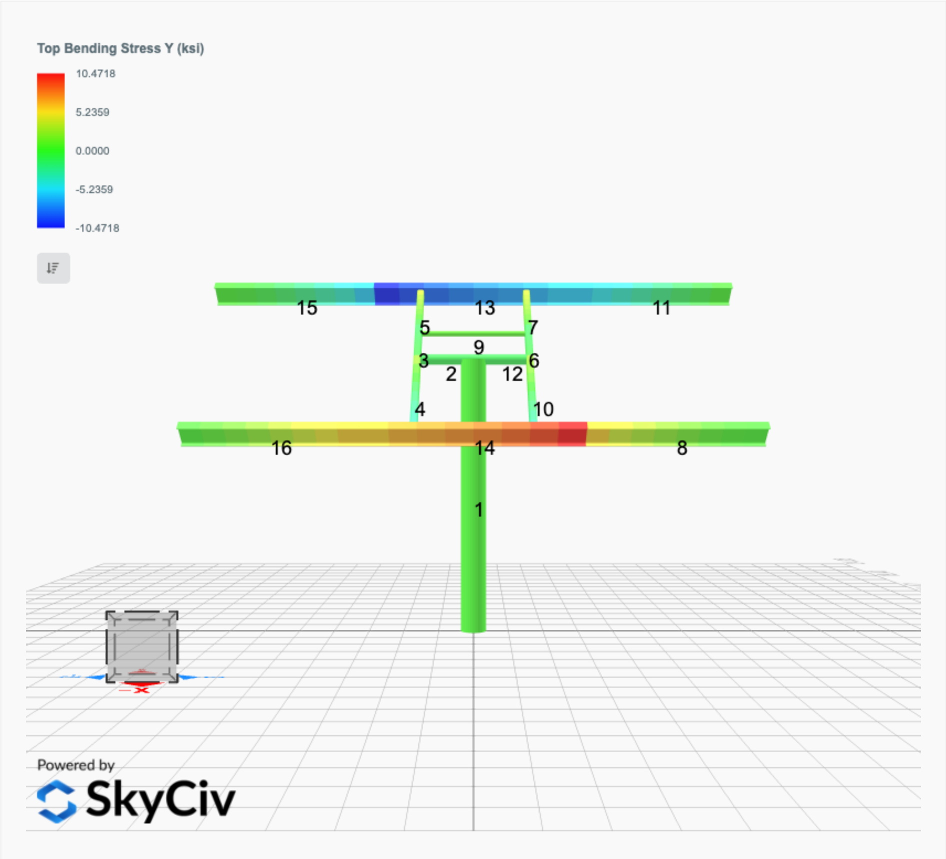
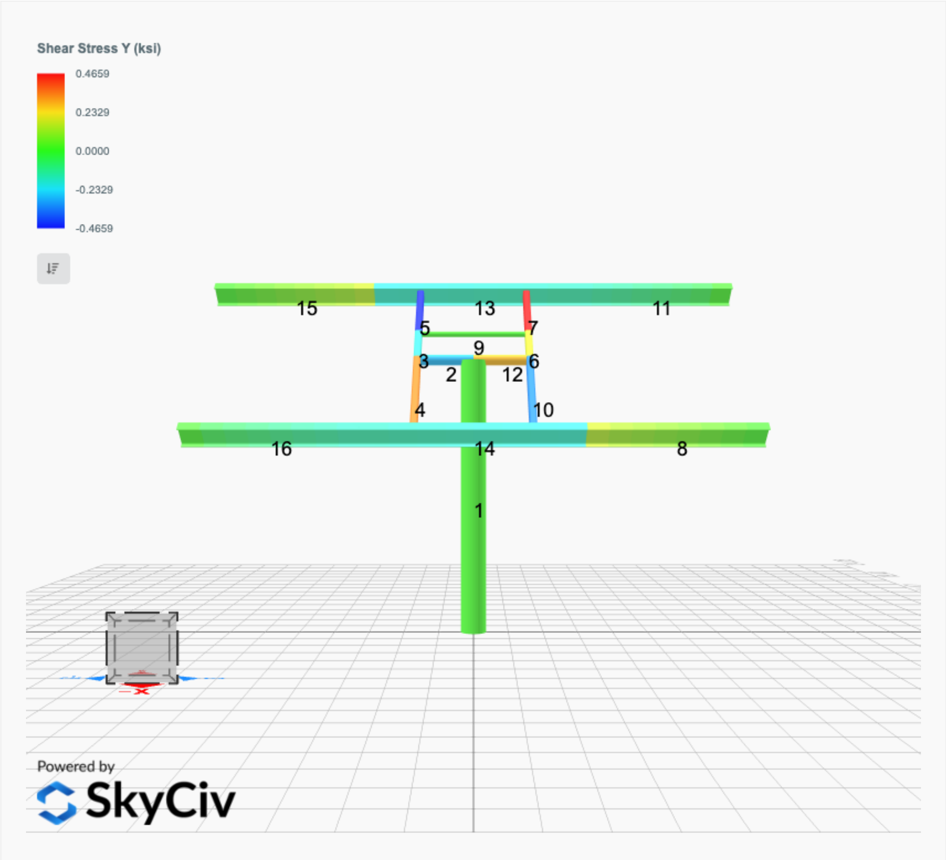
16
1
Powered by
SkyCiv

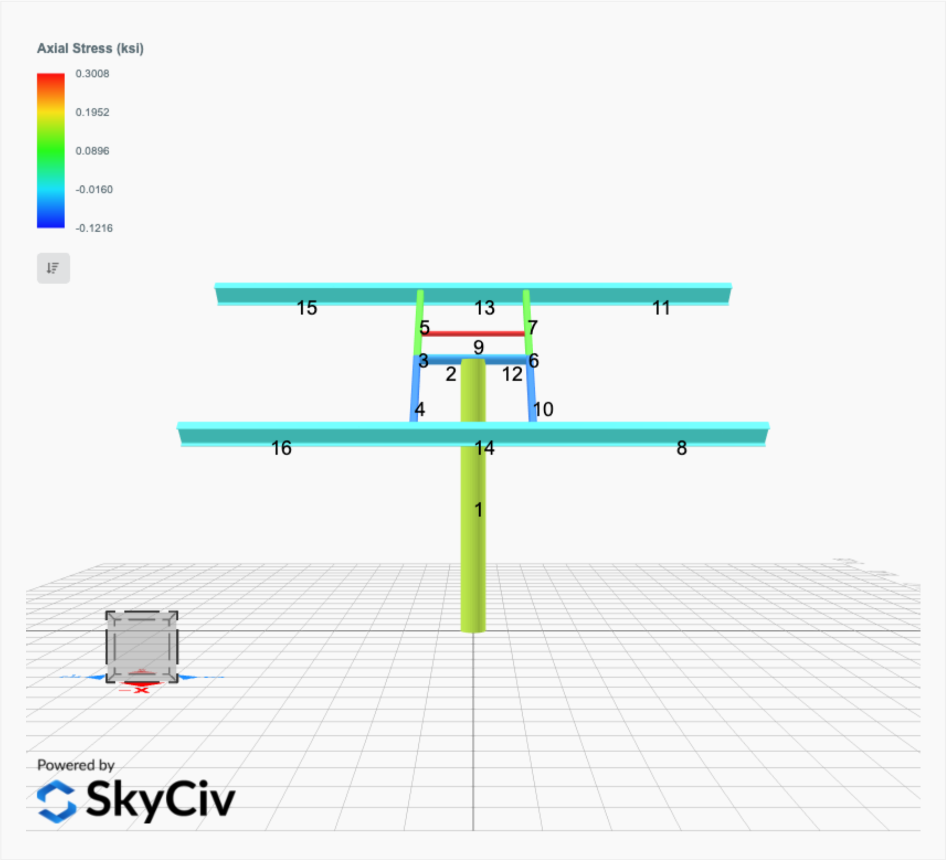
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
10.4718
5.2359
0.0000
-5.2359
-10.4718
15
13
11
5
7
9
3
2
12
6
4
10
16
14
8
1
Powered by
SkyCiv


Shear Stress Y (ksi)
0.4659
0.2329
0.0000
-0.2329
-0.4659
15
13
11
5
7
9
3
2
12
6
4
10
16
14
8
1
Powered by
SkyCiv


Axial Stress (ksi)
0.3008
0.1952
0.0896
-0.0160
-0.1216
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	2.3650	0.0000	0.0000	-0.0000	0.0247
ULS: 2. D + L	0.0000	2.3650	0.0000	0.0000	-0.0000	0.0247
ULS: 3. D + (S or Lr or R)	0.0000	4.1296	0.0000	0.0000	-0.0000	0.0278
ULS: 3. D + (S or Lr or R)	0.0000	2.3650	0.0000	0.0000	-0.0000	0.0247
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.6884	0.0000	0.0000	-0.0000	0.0270
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.3650	0.0000	0.0000	-0.0000	0.0247
ULS: 5b. D + 0.7E	0.0000	2.3650	0.0000	0.0000	-0.0000	0.0247
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.6884	0.0000	0.0000	-0.0000	0.0270
ULS: 8. 0.6D + 0.7E	0.0000	1.4190	0.0000	0.0000	-0.0000	0.0148
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.7272	8.0922	-0.0000	0.0000	-0.0000	57.7994
ULS: 5a. D + 0.6W_Wind downforce Case B only	-5.7272	8.0922	-0.0000	0.0000	-0.0000	57.7994
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.1431	-1.7781	-0.0000	0.0000	-0.0000	-38.4345
ULS: 5a. D + 0.6W_Wind uplift Case B only	3.6557	-1.2906	-0.0000	0.0000	-0.0000	-45.3158
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2954	7.9838	-0.0000	0.0000	-0.0000	43.3581
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2954	7.9838	-0.0000	0.0000	-0.0000	43.3581
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1073	0.5811	-0.0000	0.0000	-0.0000	-28.8173
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7417	0.9467	-0.0000	0.0000	-0.0000	-33.9783
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.2954	6.6604	-0.0000	0.0000	-0.0000	43.3557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-4.2954	6.6604	-0.0000	0.0000	-0.0000	43.3557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.1073	-0.7423	-0.0000	0.0000	-0.0000	-28.8197
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.7417	-0.3767	-0.0000	0.0000	-0.0000	-33.9807
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.7272	7.1462	-0.0000	0.0000	-0.0000	57.7895
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-5.7272	7.1462	-0.0000	0.0000	-0.0000	57.7895
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.1431	-2.7241	-0.0000	0.0000	-0.0000	-38.4443
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	3.6557	-2.2367	-0.0000	0.0000	-0.0000	-45.3257

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	13.2656
Shear X	-9.5453
Shear Z	-0.0000
Moment X	-0.0000
Moment Z	96.7449

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	8.0922
Shear X	-5.7272
Shear Z	-0.0000
Moment X	0.0000
Moment Z	57.7994

Project Details

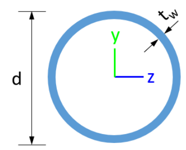
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

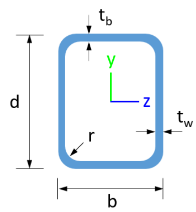
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

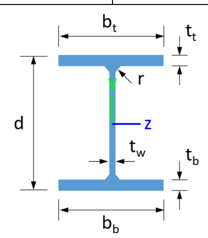
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
11	10in Pipe Sch 40	10.75	0.36					



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



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	62.42	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	11	20.28	20.28	9.66	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.60,1.67,1.67,1.66,1.66	300	200	1
5	17	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
6	17	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	20	12.60	12.60	6.00	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	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.67,1.66,1.65,1.67,1.67,1.66,1.65,1.67,1.67,1.65,1.69,1.67,1.67,1.66,1.60,1.67,1.67,1.66,1.66	300	200	1
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12	6	1.30	1.30	2.00	-	300	200	1
13	20	1.14	1.14	1.75	1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20	300	200	1
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17	20	2.60	2.60	4.00	1.04,1.05,1.04,1.05,1.04,1.04,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.04,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05	300	200	1
18	20	1.14	1.14	1.75	1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20	300	200	1
19	20	2.60	2.60	4.00	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.05,1.04,1.04,1.04,1.05,1.05,1.04,1.04,1.05,1.03,1.04,1.04,1.04,1.04	300	200	1
20	20	1.14	1.14	1.75	1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20,1.20	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	535.87	388.86	147.68	147.68	160.76	160.76

2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	21.54	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	21.54	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	143.45	46.90	6.46	56.26	44.91
14	159.30	143.45	46.90	6.46	56.26	44.91
15	159.30	21.54	46.90	6.46	56.26	44.91
16	159.30	21.54	46.90	6.46	56.26	44.91
17	159.30	132.71	44.63	6.46	56.26	44.91
18	159.30	143.45	46.90	6.46	56.26	44.91
19	159.30	132.71	44.20	6.46	56.26	44.91
20	159.30	143.45	46.90	6.46	56.26	44.91

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	0.034	0.655	0.000	0.059	0.000	0.672	#13	0.331	Not Required	Pass
2	0.004	0.401	0.333	0.085	0.063	0.736	#13	0.036	Not Required	Pass
3	0.006	0.770	0.038	0.077	0.006	0.786	#13	0.046	Not Required	Pass
4	0.006	0.688	0.149	0.069	0.031	0.742	#13	0.082	Not Required	Pass
5	0.006	0.478	0.155	0.076	0.041	0.504	#13	0.076	Not Required	Pass
6	0.006	0.770	0.038	0.077	0.006	0.786	#13	0.046	Not Required	Pass
7	0.006	0.478	0.155	0.076	0.041	0.504	#13	0.076	Not Required	Pass
8	0.000	0.145	0.246	0.040	0.012	0.342	#21	Not Required	Not Required	Pass
9	0.020	0.059	0.065	0.001	0.000	0.129	#13	0.206	Not Required	Pass
10	0.006	0.688	0.149	0.069	0.031	0.742	#13	0.082	Not Required	Pass
11	0.000	0.162	0.246	0.045	0.012	0.350	#21	Not Required	Not Required	Pass
12	0.004	0.401	0.333	0.085	0.063	0.736	#13	0.036	Not Required	Pass
13	0.000	0.270	0.410	0.058	0.015	0.585	#21	Not Required	Not Required	Pass
14	0.000	0.241	0.410	0.052	0.015	0.571	#21	Not Required	Not Required	Pass
15	0.000	0.162	0.246	0.045	0.012	0.350	#21	Not Required	Not Required	Pass
16	0.000	0.145	0.246	0.040	0.012	0.342	#21	Not Required	Not Required	Pass
17	0.007	0.296	0.072	0.015	0.004	0.338	#13	0.133	Not Required	Pass
18	0.000	0.270	0.410	0.058	0.015	0.585	#21	Not Required	Not Required	Pass
19	0.007	0.270	0.084	0.013	0.004	0.323	#13	0.199	Not Required	Pass
20	0.000	0.241	0.410	0.052	0.015	0.571	#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	Moment of inertia about the Y axes

I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

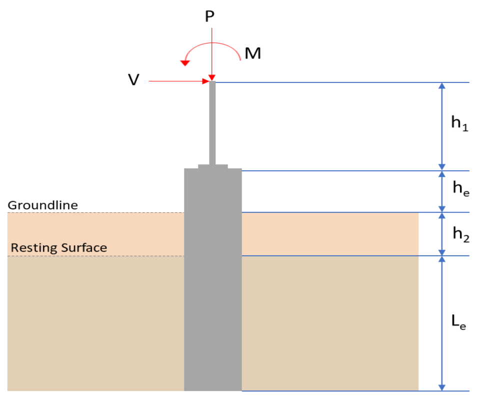
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36\ \mathrm{in}$ - Pile diameter
$L = 15\ \mathrm{ft}$ - 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)	8.092	13.266
V_x (kip)	-5.727	-9.545
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	57.799	96.745

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,

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

$$H_o = \frac{(-5.727\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -1.909\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.799 \text{ kipft}) + ((-5.727 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 19.266 \text{ kipft/ft}$$

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:
$L_{e,x} = 9.1697 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(9.1697 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 9.17 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (15 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 15 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(9.17 \text{ ft})}{(15 \text{ ft})}$$

$$Ratio = 0.61133$$

Status: **PASS**
Ratio: **0.610**

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(8.092 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.1448 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.1448 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.57239$$

Status: **PASS**
Ratio: **0.570**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(15 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -1.909\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 19.266\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (19.266\ \mathrm{kipft/ft}) \times (15\ \mathrm{ft})) + \left(3 \times (-1.909\ \mathrm{kip/ft}) \times (15\ \mathrm{ft})^2\right)}{(6 \times (19.266\ \mathrm{kipft/ft})) + (4 \times (-1.909\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))}$$

$$a = 10.622\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (19.266\ \mathrm{kipft/ft})) + (3 \times (-1.909\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]^{\ 2}}{(15\ \mathrm{ft})^2 \times [(3 \times (19.266\ \mathrm{kipft/ft})) + (2 \times (-1.909\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]}$$

$$p = 0.77335\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (19.266\ \mathrm{kipft/ft})) + ((-1.909\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]}{(15\ \mathrm{ft})^2}$$

$$s = 0.4146\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(10.622\ \mathrm{ft})}{2}$$

$$p_a = 0.79666\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.77335\ \mathrm{kip/ft^2}\right)}{\left(0.79666\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.97075$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (15\ \mathrm{ft})$$

$$p_s = 2.25\ \mathrm{kip/ft^2}$$

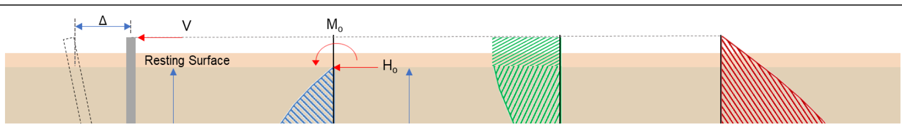
$Ratio$ - Lateral soil capacity

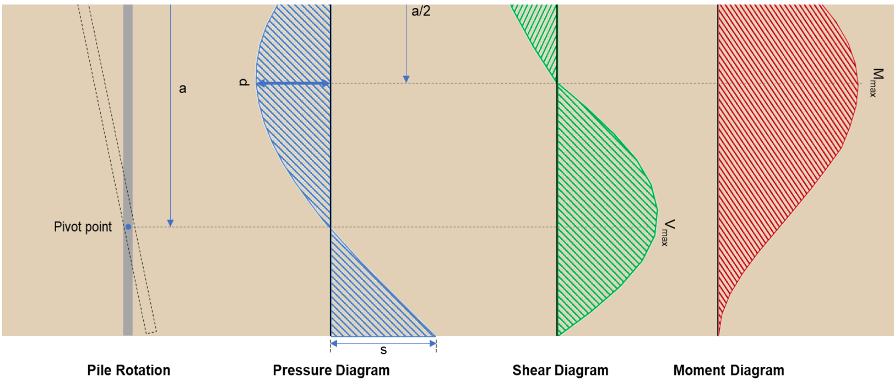
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.4146\ \mathrm{kip/ft^2}\right)}{\left(2.25\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.18427$$

Status: **PASS**
Ratio: **0.180**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-9.545 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -3.1817 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(96.745 \text{ kipft}) + ((-9.545 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 32.248 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(32.248 \text{ kipft/ft})}{(-3.1817 \text{ kip/ft})}$$

$$E = 10.136 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (32.248 \text{ kipft/ft}) \times (15 \text{ ft})) + \left(3 \times (-3.1817 \text{ kip/ft}) \times (15 \text{ ft})^2\right)}{(6 \times (32.248 \text{ kipft/ft})) + (4 \times (-3.1817 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.621 \text{ ft}$$

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

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

$$V_{max} = ((-3.1817 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.136 \text{ ft})}{(15 \text{ ft})} + 3\right) \times \left(\frac{(10.621 \text{ ft})}{(15 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.136 \text{ ft})}{(15 \text{ ft})} + 2\right) \times \left(\frac{(10.621 \text{ ft})}{(15 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 17.745 \text{ kip}$$

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

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

$$M_{max} = ((-3.1817 \text{ kip/ft}) \times (36 \text{ in}) \times (15 \text{ ft})) \times \left[\left(\frac{(10.136 \text{ ft})}{(15 \text{ ft})} + \frac{(10.621 \text{ ft})}{2 \times (15 \text{ ft})}\right) - \left[\left(\frac{4 \times (10.136 \text{ ft})}{(15 \text{ ft})} + 3\right) \times \left(\frac{(10.621 \text{ ft})}{2 \times (15 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.136 \text{ ft})}{(15 \text{ ft})} + 2\right) \times \left(\frac{(10.621 \text{ ft})}{2 \times (15 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 120.26 \text{ kipft}$$

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

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.266 \text{ kip})}{(0.65)\times(0.85)} - \left(0.85 \times (3 \text{ ksi}) \times \left(1017.9 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -44.762 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-44.762 \text{ in}^2\right), \left(0.0018 \times \left(1017.9 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties,

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

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

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

Summary:

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.85\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 1492.5\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(13.266\text{ kip})}{(1492.5\text{ kip})}$$ $$Ratio = 0.0088885$$	Status: **PASS** Ratio: **0.010**

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.71796$$

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

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,max} = 203.86\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $P = 13.266\text{ kip} \rightarrow 13266\text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + \frac{(13266\text{ lbf})}{6 \times \left(1017.9\text{ in}^2\right)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,a} = 83.795\text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,b} = 237.06\text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(203.86\text{ kip}), (83.795\text{ kip}), (237.06\text{ kip})\right]$$

$$V_c = 83.795 \text{ kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 454.3 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(454.3 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((83.795 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 79.277 \text{ kip}$$

Considering x-direction:

$V_{max} = 17.745 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(17.745 \text{ kip})}{(79.277 \text{ kip})}$$

$$Ratio = 0.22383$$

Status: **PASS**
Ratio: **0.220**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi\, D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 4580.442in^3$$

$$\phi M_{n,1} = 67.947kipft$$

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

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times \left(4580.4 \text{ in}^3\right)$$

$$\phi M_{n,2} = 632.67 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$$

$$\phi M_n = 67.947 \text{ kipft}$$

Considering x-direction:

$M_{max} = 120.26 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(120.26 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 1.7699$$

Status: **FAIL**
Ratio: **1.770**