Your Project Calculations

Project Name: Van Bogart

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Van%20Bogart&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/3_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=sIXfPmYo6YYIR0KjcONKNAEhNUolbZgXGhFKUHCTOXyolvhP2GX4Mdt4CZdRuU2p

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-57-L-4Hx3W-K37J
Duty Classification:	SD
Module Width:	44.60 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	50
Front Edge Clearance:	3
Total Array Height at Tilt:	14.45 ft
Total Frame Length:	17.00 ft
Frame Weight:	625 lbs
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	17.20 ft
Rail Length:	180.40 in
Rail Spacing:	2.82 ft
Rail Check:	Not Checked

Support Specifications

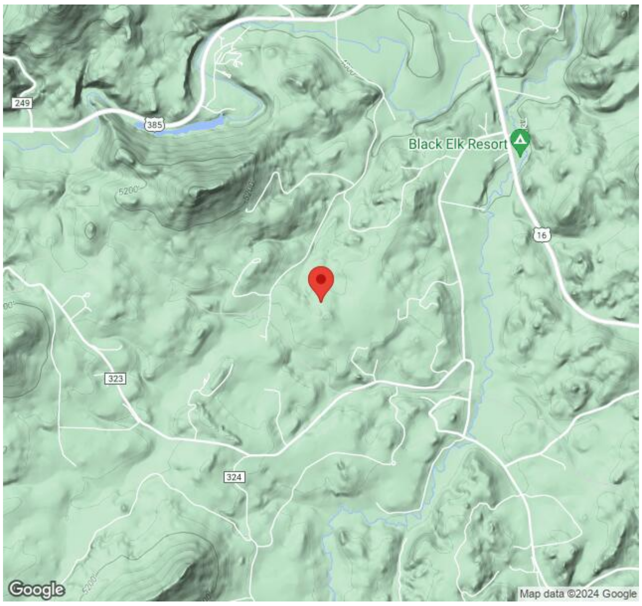
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.76 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 6.25 ft
Foundation Volume:	1.636 y^3
Foundation Result:	PASSED
Mount Twist:	0.000002 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	23960 Palmer Gulch Rd, Hill City, SD 57745, USA
Wind Speed:	90 mph
Snow Load:	30 psf
Design Uplift Pressure:	0.008529 ksf
Design Downforce Pressure:	-0.008529 ksf
Design Snow Pressure:	0.006598 ksf



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

AutoDesigner Input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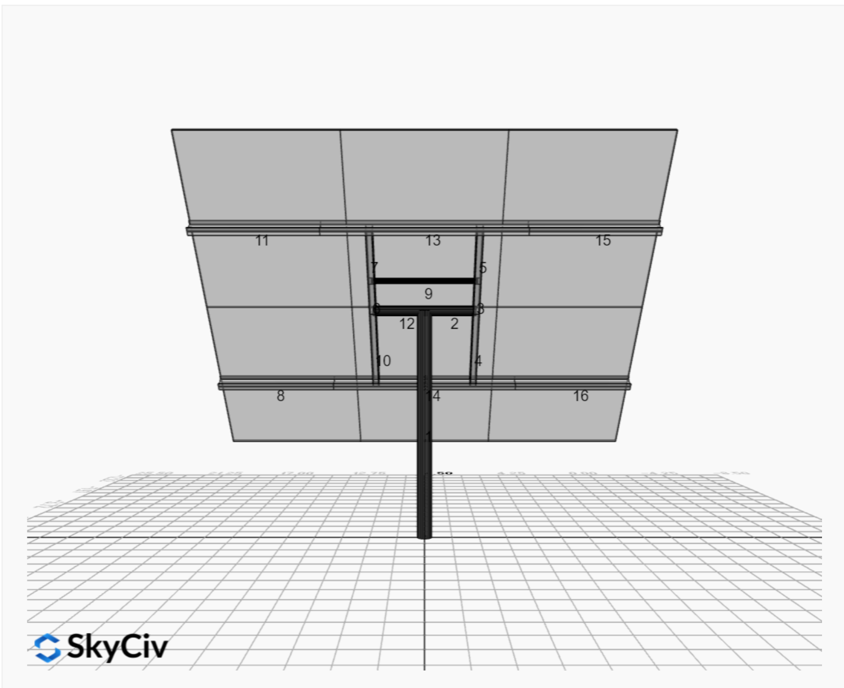
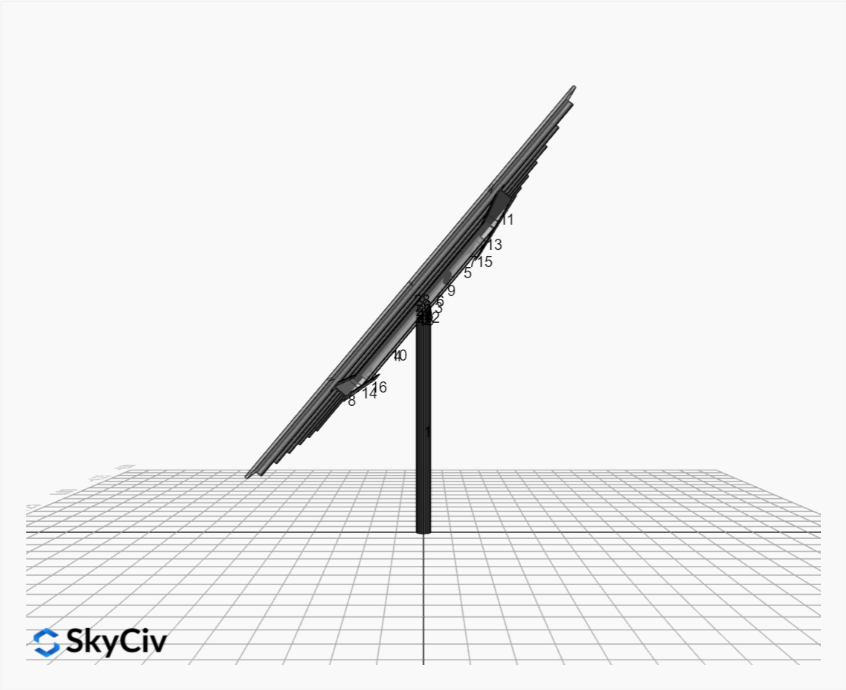
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only

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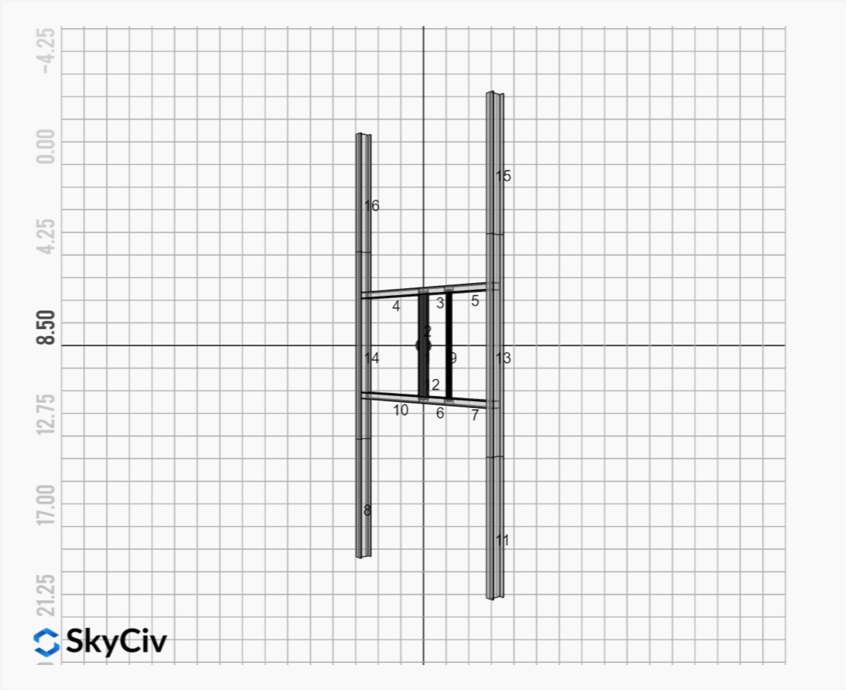
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0.00
4.25
8.50
12.75
17.00
21.25
16
15
4
3
5
14
9
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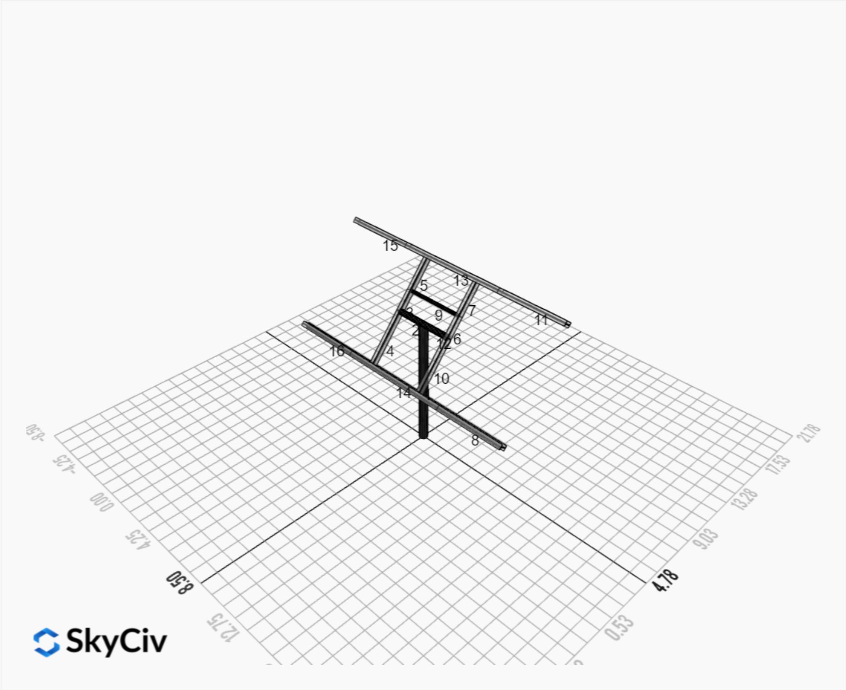
2
10
6
7
8
11
SkyCiv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5
13
9
7
11
16
4
10
14
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9.03
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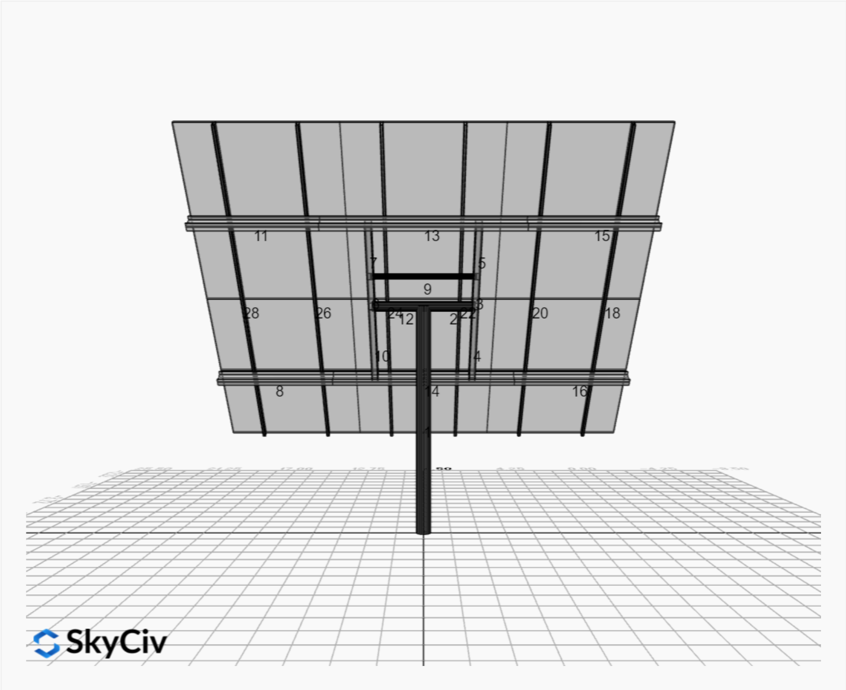
17.53
21.78
0.53
12.75
4.25
0.00
-4.25
SkyCiv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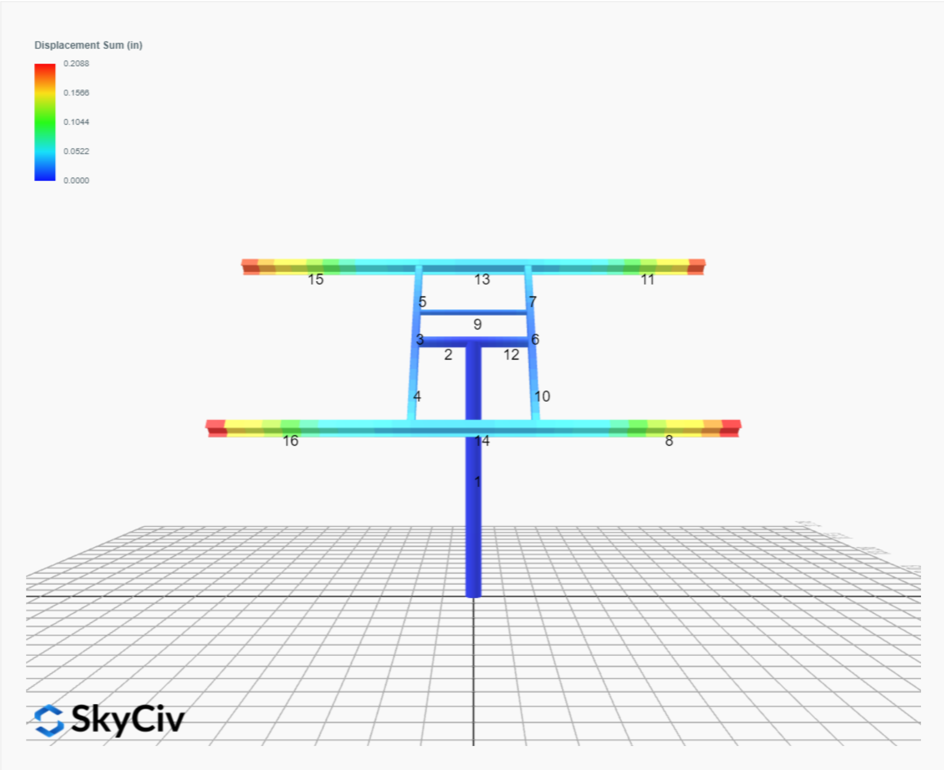
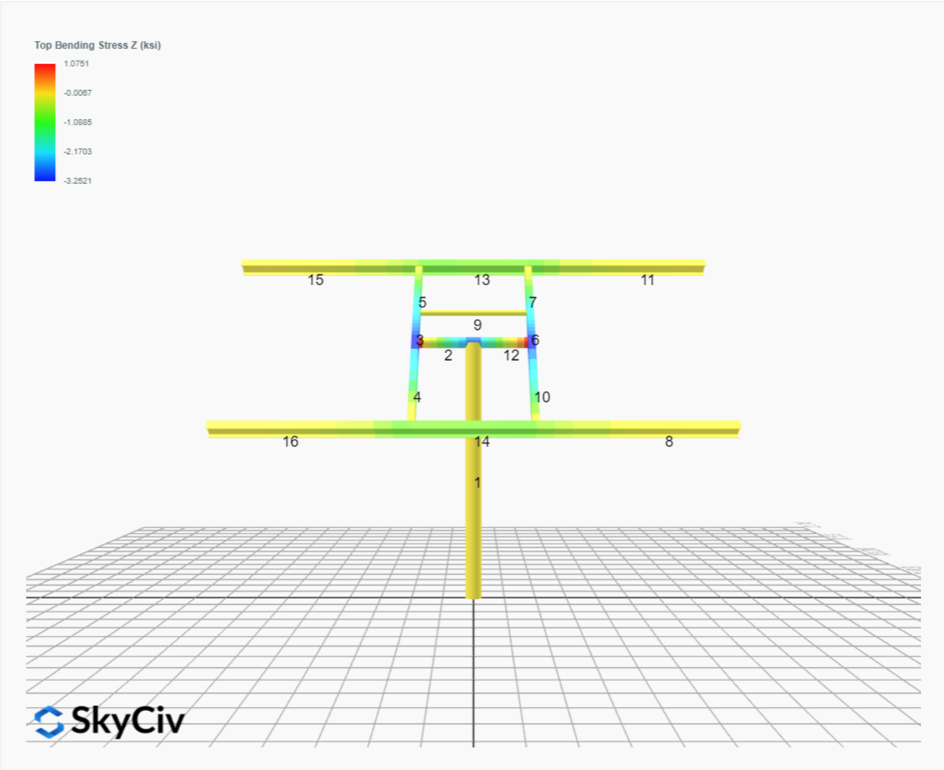
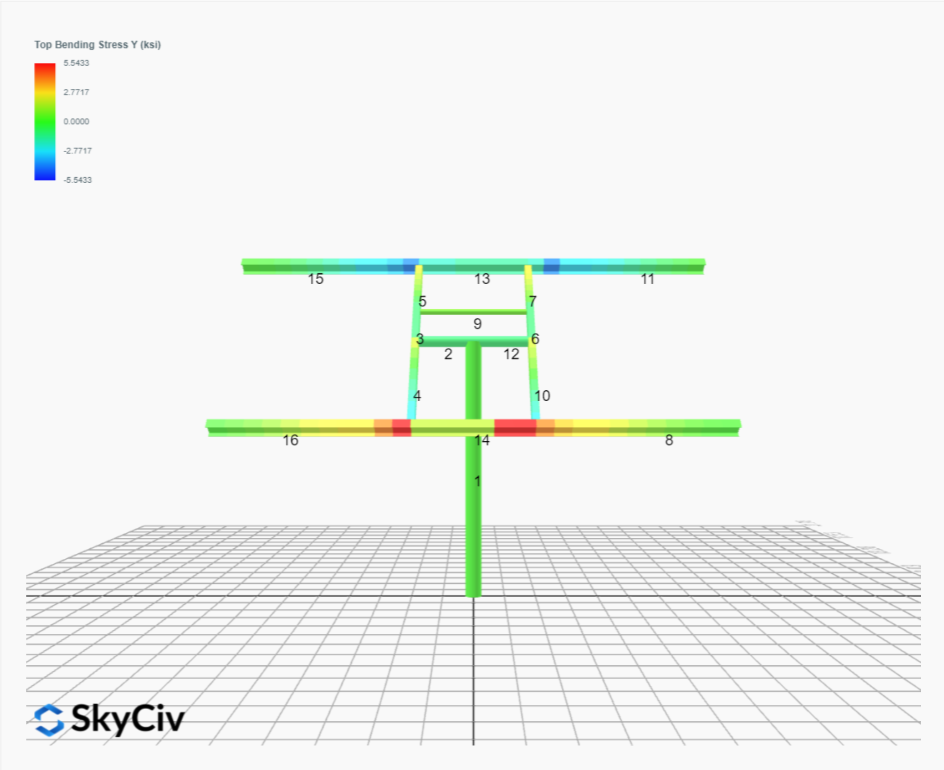
7
5
9
28
26
24
12
2
22
20
18
10
4
8
14
16
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
5.5433
2.7717
0.0000
-2.7717
-5.5433
15
13
11
5
7
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6
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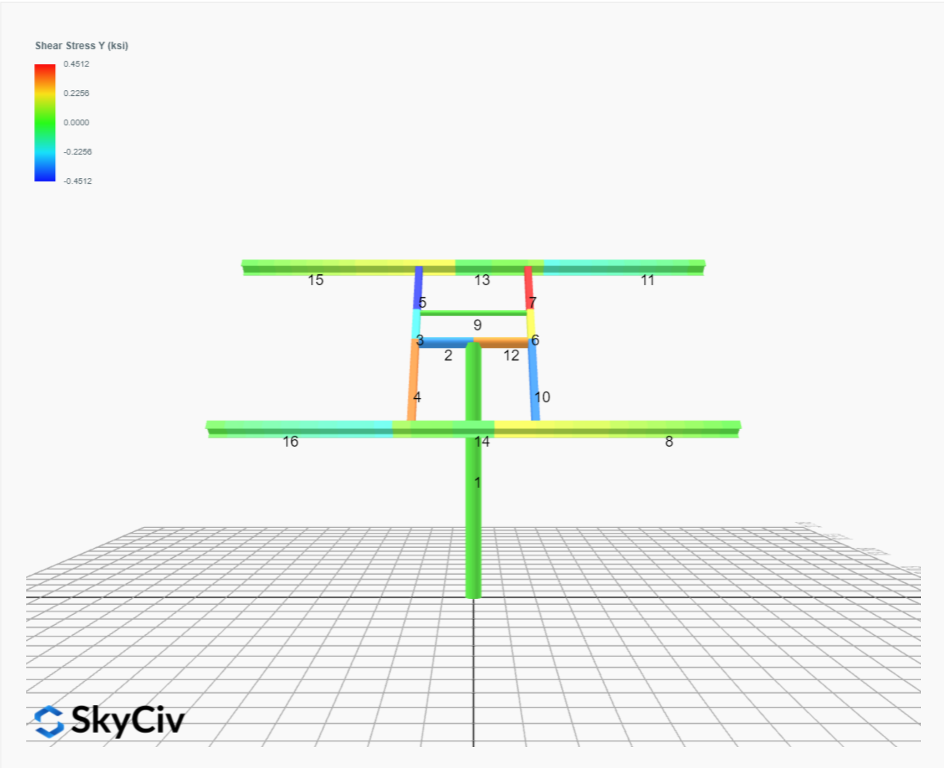
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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0.2256
0.0000
-0.2256
-0.4512
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13
11
5
7
9
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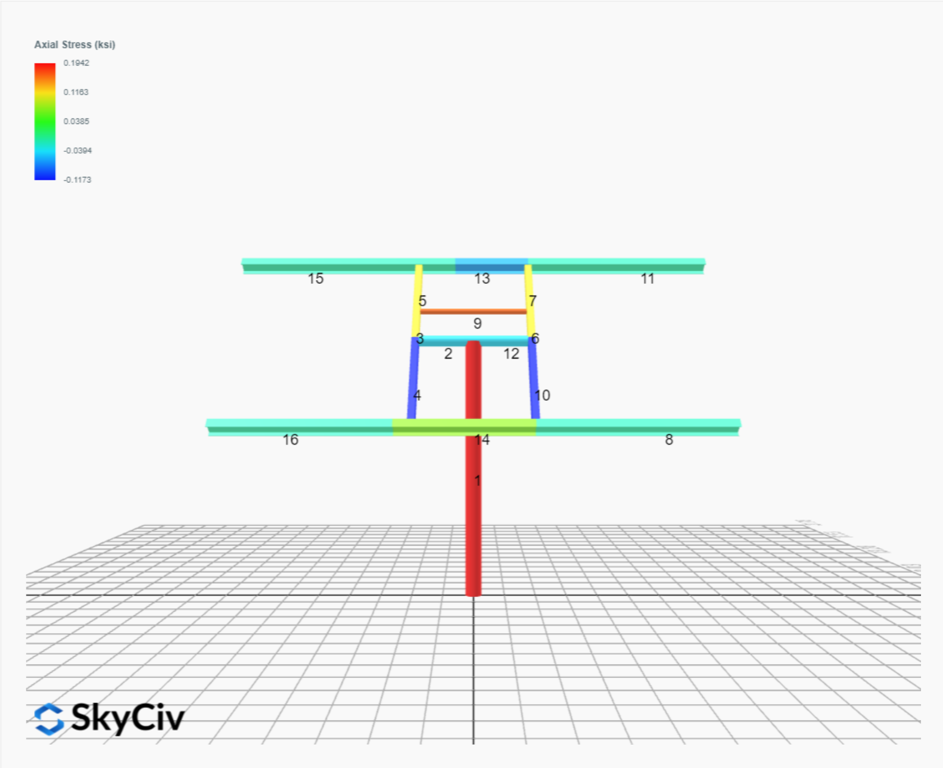
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
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-0.0394
-0.1173
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv

Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 2. D + L	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 3. D + (S or Lr or R)	0.0000	2.9227	0.0000	0.0000	-0.0000	0.0184
ULS: 3. D + (S or Lr or R)	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.6518	0.0000	0.0000	-0.0000	0.0178
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 5b. D + 0.7E	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.6518	0.0000	0.0000	-0.0000	0.0178
ULS: 8. 0.6D + 0.7E	0.0000	1.1033	0.0000	0.0000	-0.0000	0.0097
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0136	2.6894	0.0000	0.0000	-0.0000	8.9508
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0136	0.9883	0.0000	0.0000	-0.0000	-8.8048
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7602	3.2896	0.0000	0.0000	-0.0000	6.7187
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.6518	0.0000	0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7602	2.0139	0.0000	0.0000	-0.0000	-6.5979
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.6518	0.0000	0.0000	-0.0000	0.0178
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7602	2.4768	0.0000	0.0000	-0.0000	6.7172
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7602	1.2010	0.0000	0.0000	-0.0000	-6.5995
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.8389	0.0000	0.0000	-0.0000	0.0162
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0136	1.9538	0.0000	0.0000	-0.0000	8.9443
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.1033	0.0000	0.0000	-0.0000	0.0097
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0136	0.2528	0.0000	0.0000	-0.0000	-8.8113
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.1033	0.0000	0.0000	-0.0000	0.0097

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	4.6496
Shear X	-1.6894
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	15.1229

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	3.2896
Shear X	-1.0136
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	8.9508

Project Details

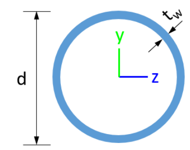
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

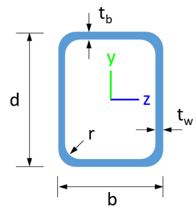
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

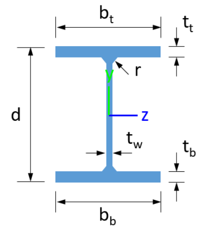
Section Dimensions



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	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	7	18.39	18.39	8.76	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.26,1.18,1.18,1.18,1.17,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.64,1.68,1.67,1.68,1.65,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.85,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.79,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.26,1.18,1.18,1.18,1.17,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.79,1.68,1.67,1.68,1.66,1.68	300	200	1
8	18	9.97	9.97	4.75	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	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.64,1.68,1.67,1.68,1.65,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.85,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	9.97	9.97	4.75	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	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
14	18	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
15	18	9.97	9.97	4.75	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	9.97	9.97	4.75	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	123.93	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	34.69	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61

11	120.60	34.69	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	18.12	6.45	30.09	45.74
14	120.60	98.23	18.10	6.45	30.09	45.74
15	120.60	34.69	23.36	6.45	30.09	45.74
16	120.60	34.69	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	0.038	0.358	0.000	0.022	0.000	0.374	#13	0.491	Not Required	Pass
2	0.003	0.209	0.121	0.052	0.020	0.318	#13	0.034	Not Required	Pass
3	0.010	0.324	0.079	0.033	0.004	0.382	#13	0.044	Not Required	Pass
4	0.009	0.322	0.258	0.033	0.031	0.464	#21	0.078	Not Required	Pass
5	0.009	0.201	0.270	0.032	0.040	0.275	#6	0.073	Not Required	Pass
6	0.010	0.324	0.079	0.033	0.004	0.382	#13	0.044	Not Required	Pass
7	0.009	0.201	0.270	0.032	0.040	0.275	#6	0.073	Not Required	Pass
8	0.000	0.053	0.140	0.017	0.008	0.189	#21	Not Required	Not Required	Pass
9	0.014	0.022	0.035	0.001	0.000	0.061	#13	0.198	Not Required	Pass
10	0.009	0.322	0.258	0.033	0.031	0.464	#21	0.078	Not Required	Pass
11	0.000	0.053	0.140	0.017	0.008	0.189	#21	Not Required	Not Required	Pass
12	0.003	0.209	0.121	0.052	0.020	0.318	#13	0.034	Not Required	Pass
13	0.005	0.131	0.263	0.024	0.011	0.379	#21	0.177	Not Required	Pass
14	0.005	0.134	0.263	0.024	0.011	0.379	#21	0.177	Not Required	Pass
15	0.000	0.053	0.140	0.017	0.008	0.189	#21	Not Required	Not Required	Pass
16	0.000	0.053	0.140	0.017	0.008	0.189	#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

N/C capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

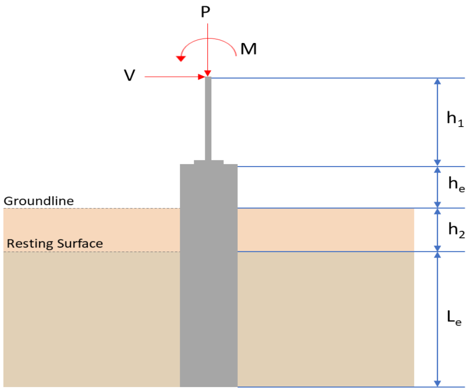
SkyCiv Foundation Design

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
$L = 6.25\ \mathrm{ft}$ - 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)	3.290	4.650
V_x (kip)	-1.014	-1.689
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	8.951	15.123

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.014\ \mathrm{kip})}{(36\ \mathrm{in})}$$

$$H_o = -0.338\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(8.951\ \text{kipft}) + ((-1.014\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 2.9837\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.7624\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(5.7624\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 5.762\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.762\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.92192$$

Status: **PASS**
Ratio: **0.920**

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

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

$$q = \frac{(3.29\ \text{kip})}{(7.0686\ \text{ft}^2)}$$

$$q = 0.46544\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.46544\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.23272$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \text{ft})}{(36\ \text{in})}$$

$$L/D = 2.0833$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.338\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.9837\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9837\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.338\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (2.9837\ \mathrm{kipft/ft})) + (4 \times (-0.338\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.3337\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.9837\ \mathrm{kipft/ft})) + (3 \times (-0.338\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (2.9837\ \mathrm{kipft/ft})) + (2 \times (-0.338\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.19991\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (2.9837\ \mathrm{kipft/ft})) + ((-0.338\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.93009\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.3337\ \mathrm{ft})}{2}$$

$$p_a = 0.32503\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.19991\ \mathrm{kip/ft^2}\right)}{\left(0.32503\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.61505$$

Status: **PASS**
Ratio: **0.620**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\ L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

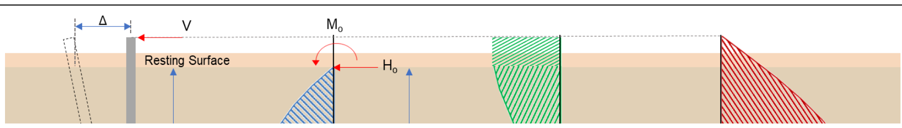
$Ratio$ - Lateral soil capacity

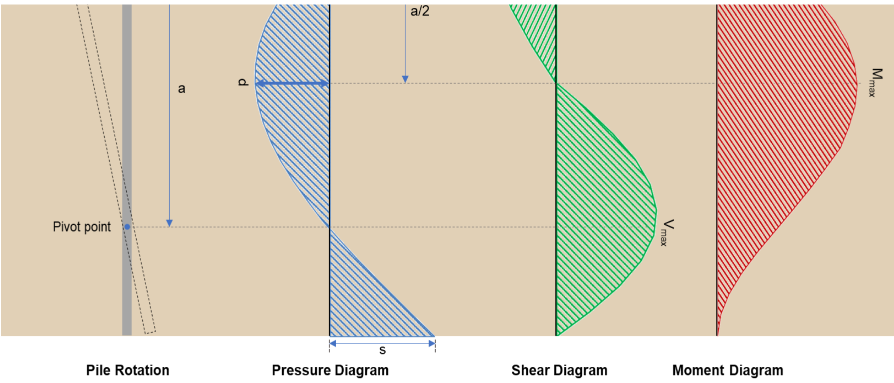
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.93009\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9921$$

Status: **PASS**
Ratio: **0.990**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.689\ \text{kip})}{(36\ \text{in})}$$

$$H_o = -0.563\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.123\ \text{kipft}) + ((-1.689\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 5.041\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.041\ \text{kipft/ft})}{(-0.563\ \text{kip/ft})}$$

$$E = 8.9538\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.041\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.563\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (5.041\ \text{kipft/ft})) + (4 \times (-0.563\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3321\ \text{ft}$$

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

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

$$V_{max} = ((-0.563\ \text{kip/ft}) \times (36\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.9538\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3321\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (8.9538\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3321\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 5.3953\ \text{kip}$$

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

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

$$M_{max} = ((-0.563\ \text{kip/ft}) \times (36\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(8.9538\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.3321\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (8.9538\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3321\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (8.9538\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3321\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 15.904 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(4.65 \text{ kip})}{(0.65)\times(0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2))\right]$$

$$A_{st,required} = -37.228 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-37.228 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

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

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

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

$$Ratio = 0.99533$$

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

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 center-to-center spacing of ties,

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

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

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

Summary:

	Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$

$$\phi P_N = 1253.9\text{ kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(4.65\text{ kip})}{(1253.9\text{ kip})}$$

$$Ratio = 0.0037084$$

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

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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,max} = 186.09\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 4.65\text{ kip} \rightarrow 4650\text{ lbf}$,
22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500\text{ psi})} + \frac{(4650\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,a} = 75.227\text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,
22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$

$$V_{c,b} = 204.04\text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(186.09\text{ kip}), (75.227\text{ kip}), (204.04\text{ kip})\right]$$

$$V_c = 75.227 \text{ kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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

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

$$V_s = MIN\left[(414.72 \text{ kip}), (38.17 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.227 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.708 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.3953 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(5.3953 \text{ kip})}{(73.708 \text{ kip})}$$

$$Ratio = 0.073198$$

Status: **PASS**
Ratio: **0.070**

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:
$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 4580.442in^3$$

$$\phi M_{n,1} = 62.027kipft$$

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

	$$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(4580.4\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 527.23\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(62.027\ \mathrm{kipft}), (527.23\ \mathrm{kipft})]$$ $$\phi M_n = 62.027\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 15.904\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(15.904\ \mathrm{kipft})}{(62.027\ \mathrm{kipft})}$$ $$Ratio = 0.25641$$	Status: **PASS** Ratio: **0.260**