Your Project Calculations

Project Name: Bill Townsend

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Bill%20Townsend&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=zaXnnAJwCdCYkHICaDi1jKdEUZD8oJUdMwzkHe1mWp4NW80JEeTEFZQ1018YS4SY

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-SD-57-L-4Hx3W-B2CC
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:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft
Foundation Volume:	3.407 y^3
Foundation Result:	PASSED
Mount Twist:	0.000005 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	North Branch, NY, USA
Wind Speed:	105 mph
Snow Load:	40 psf
Design Uplift Pressure:	0.019990 ksf
Design Downforce Pressure:	-0.019990 ksf
Design Snow Pressure:	0.008797 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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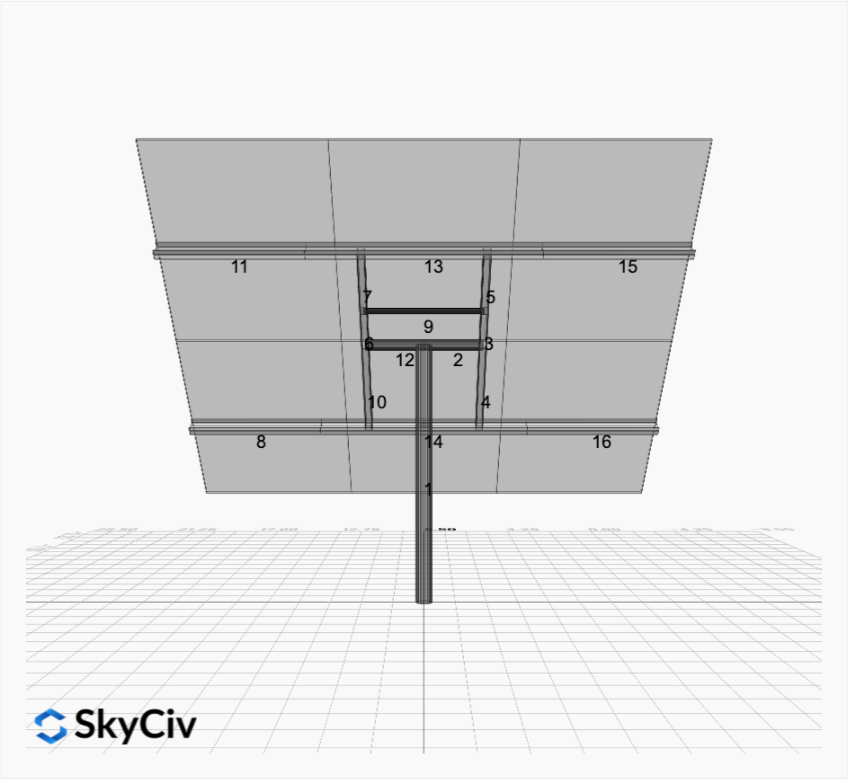
Design Notes:

- 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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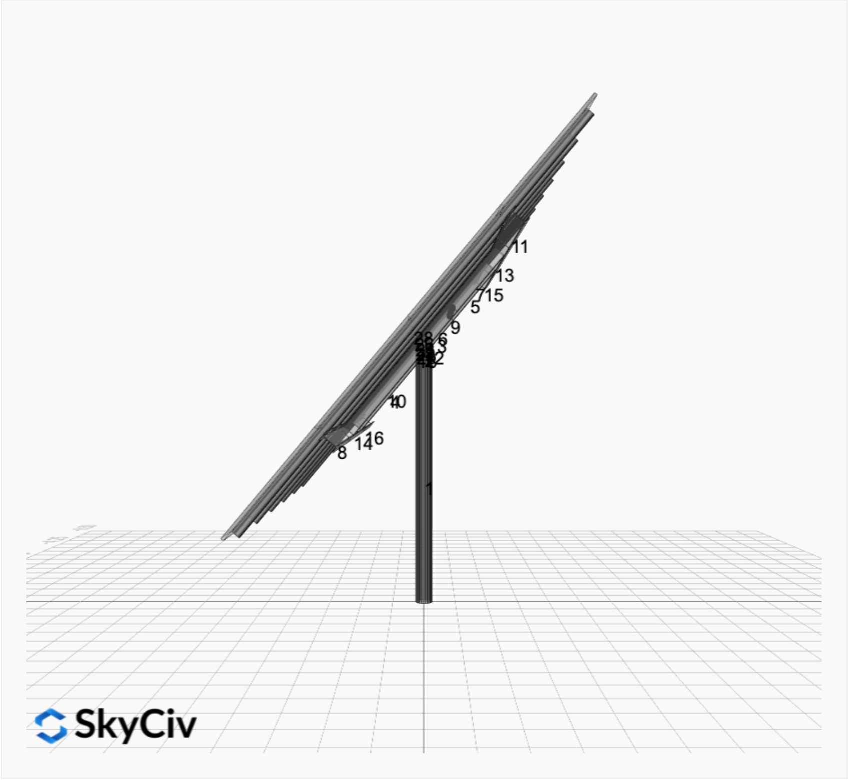
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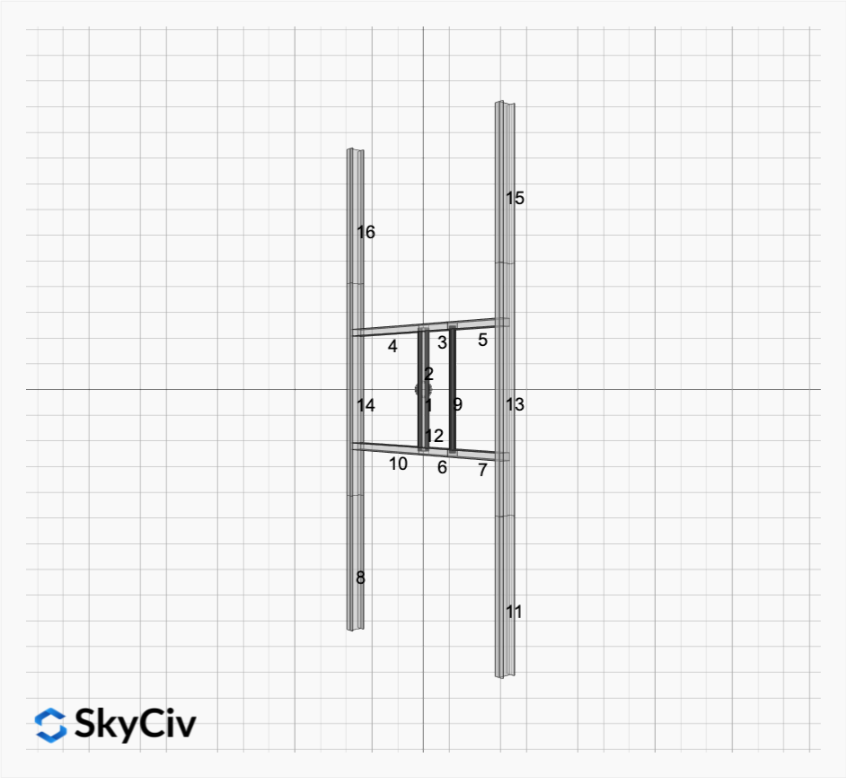
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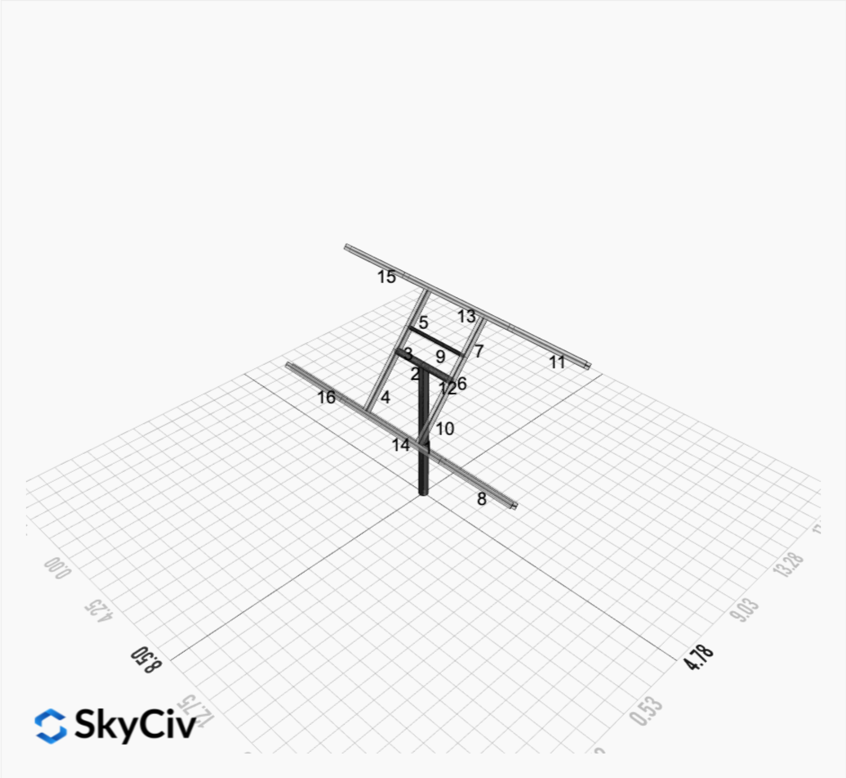
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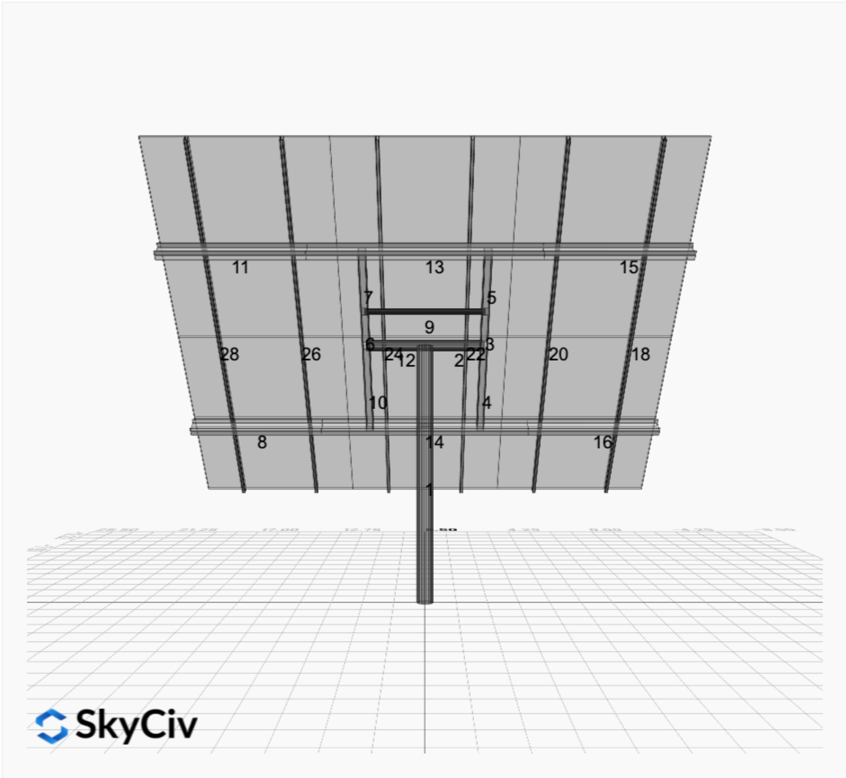
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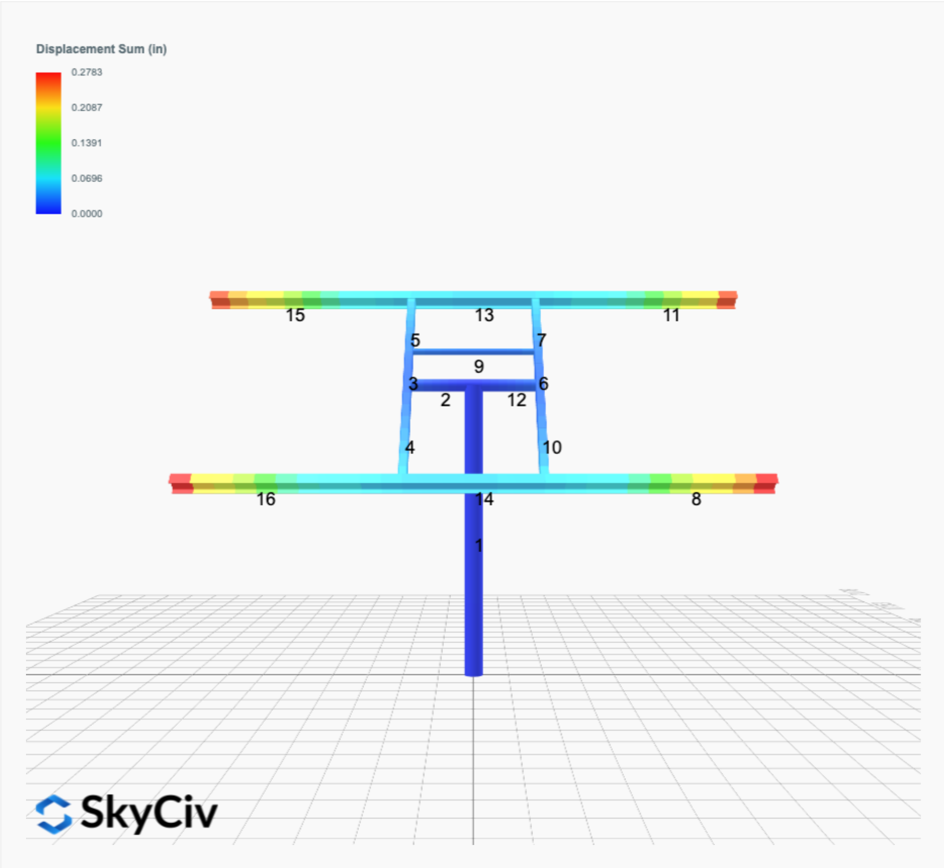
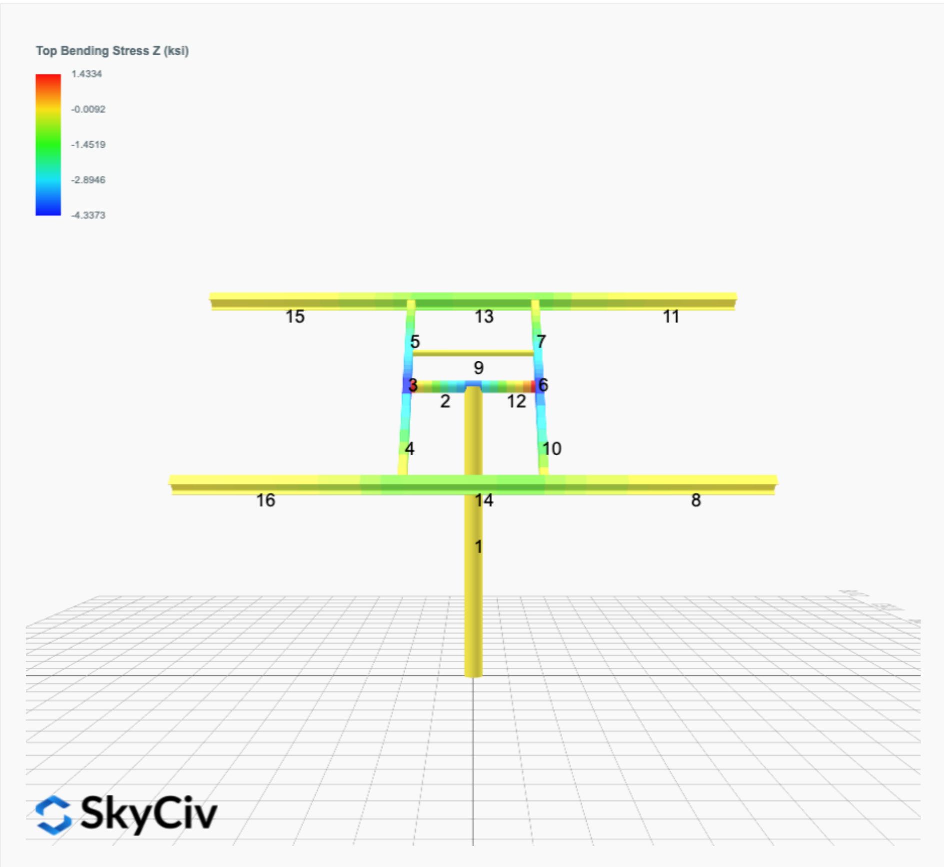
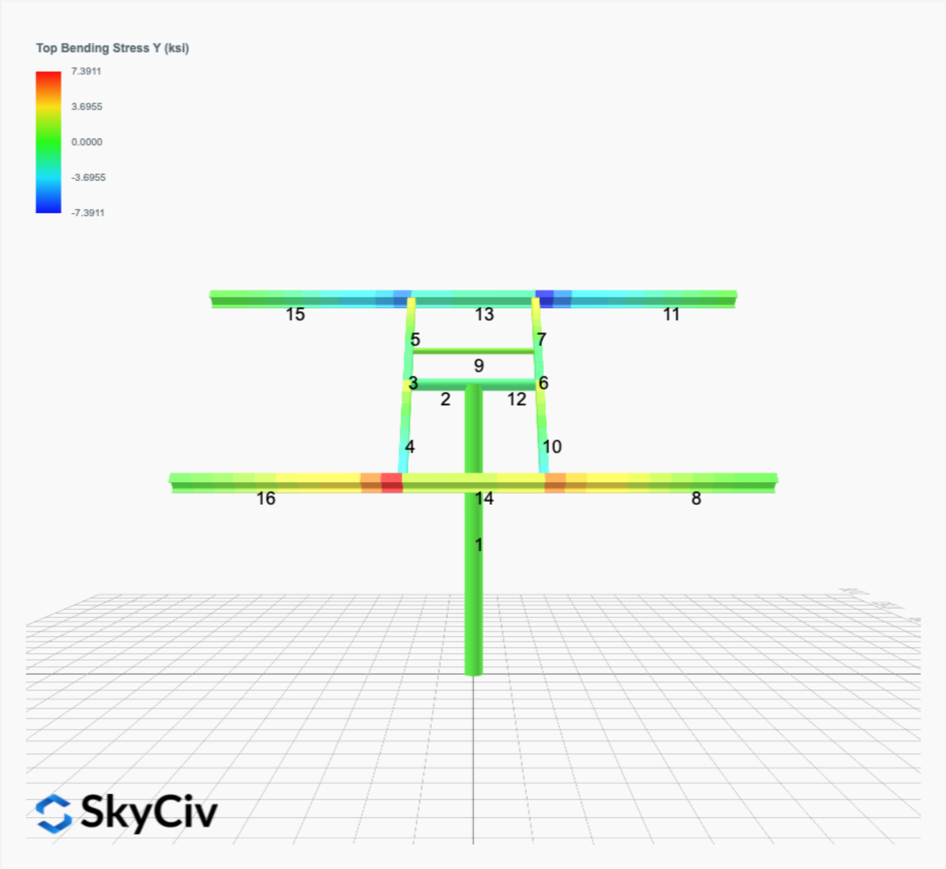
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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-3.6955
-7.3911
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13
11
5
7
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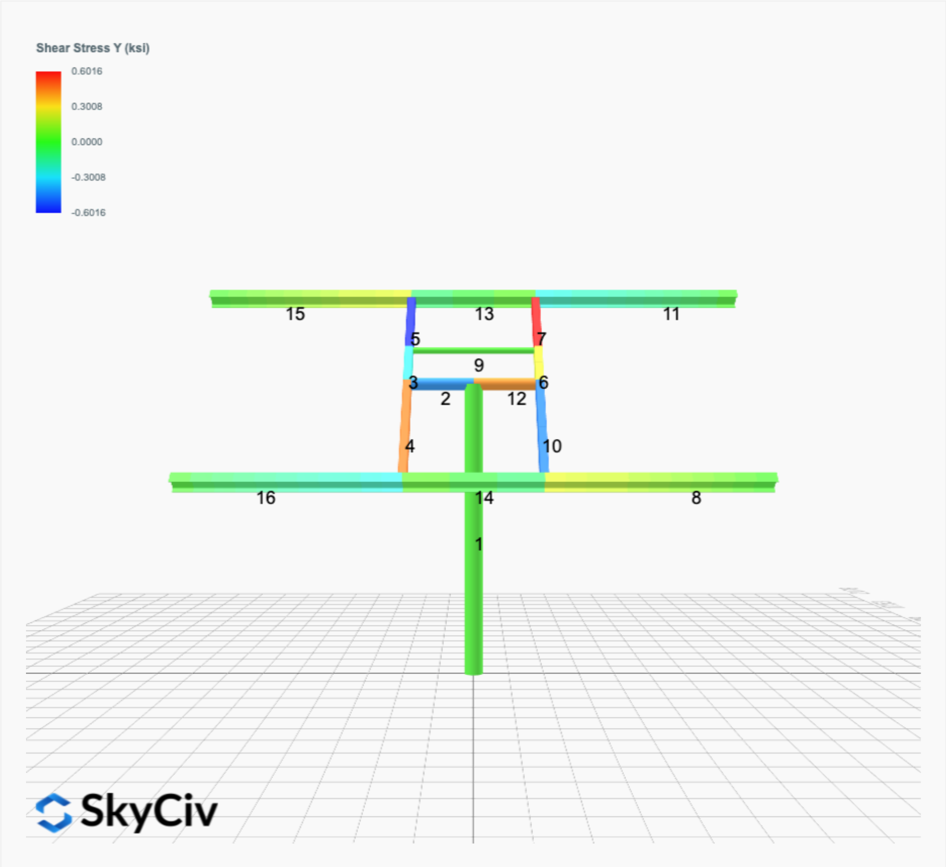
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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0.0000
-0.3008
-0.6016
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13
11
5
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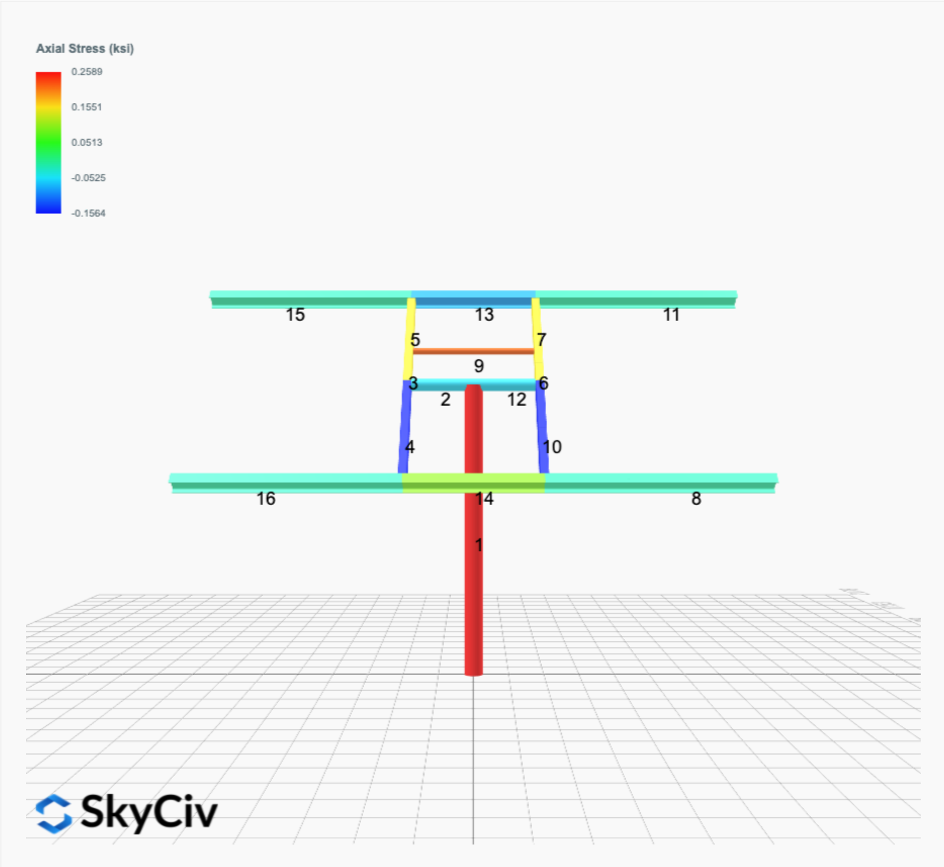
3
6
2
12
4
10
16
14
8
1
SkyCiv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-0.0525
-0.1564
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv

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	3.2840	0.0000	0.0000	-0.0000	0.0200
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	2.9227	0.0000	0.0000	-0.0000	0.0191
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	2.9227	0.0000	0.0000	-0.0000	0.0191
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	-2.3758	3.8324	0.0000	0.0000	-0.0000	21.1411
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	2.3758	-0.1546	0.0000	0.0000	-0.0000	-20.4846
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	-1.7818	4.4179	0.0000	0.0000	-0.0000	15.8627
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.9227	0.0000	0.0000	-0.0000	0.0191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.7818	1.4276	0.0000	0.0000	-0.0000	-15.3566
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.9227	0.0000	0.0000	-0.0000	0.0191
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7818	3.3340	0.0000	0.0000	-0.0000	15.8599
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	1.7818	0.3437	0.0000	0.0000	-0.0000	-15.3594
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	-2.3758	3.0968	0.0000	0.0000	-0.0000	21.1346
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	2.3758	-0.8902	0.0000	0.0000	-0.0000	-20.4911
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	6.2517
Shear X	-3.9596
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	35.7603

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	4.4179
Shear X	-2.3758
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	21.1411

Project Details

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

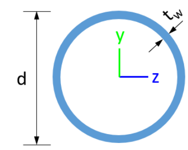
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

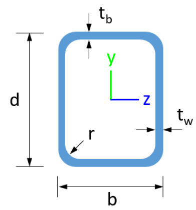
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

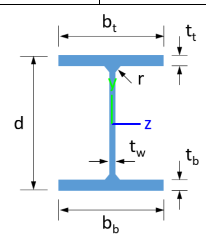
Section Dimensions



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	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.30,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.95,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.87,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.30,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.87,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	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	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.95,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	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	$\Phi_t P_n$ (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	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	84.03	18.15	6.45	30.09	45.74
14	120.60	84.03	18.13	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	0.050	0.845	0.000	0.053	0.000	0.871	#13	0.491	Not Required	Pass
2	0.004	0.313	0.253	0.071	0.046	0.567	#13	0.034	Not Required	Pass
3	0.011	0.591	0.091	0.059	0.004	0.653	#13	0.044	Not Required	Pass
4	0.010	0.588	0.300	0.059	0.036	0.711	#13	0.078	Not Required	Pass
5	0.011	0.366	0.314	0.059	0.046	0.416	#13	0.073	Not Required	Pass
6	0.011	0.591	0.091	0.059	0.004	0.653	#13	0.044	Not Required	Pass
7	0.011	0.367	0.314	0.059	0.046	0.416	#13	0.073	Not Required	Pass
8	0.000	0.097	0.163	0.032	0.010	0.238	#21	Not Required	Not Required	Pass
9	0.016	0.042	0.060	0.001	0.000	0.107	#13	0.198	Not Required	Pass
10	0.010	0.588	0.300	0.059	0.036	0.711	#13	0.078	Not Required	Pass
11	0.000	0.097	0.163	0.032	0.010	0.238	#21	Not Required	Not Required	Pass
12	0.004	0.313	0.253	0.071	0.046	0.567	#13	0.034	Not Required	Pass
13	0.006	0.240	0.306	0.043	0.013	0.485	#21	0.177	Not Required	Pass
14	0.007	0.244	0.306	0.043	0.013	0.485	#21	0.177	Not Required	Pass
15	0.000	0.097	0.163	0.032	0.010	0.238	#21	Not Required	Not Required	Pass
16	0.000	0.097	0.163	0.032	0.010	0.238	#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

NC	capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

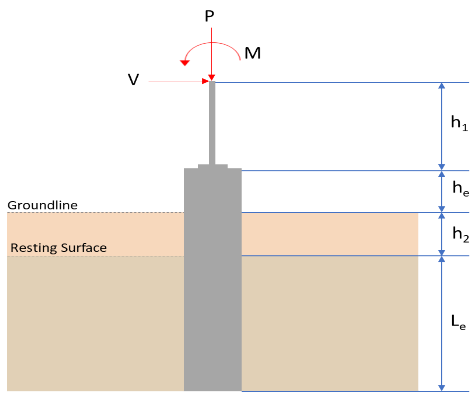
SkyCiv Foundation Design

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
$L = 5.75\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.418	6.252
V_x (kip)	-2.376	-3.960
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	21.141	35.760

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.376\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.37834\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.141 \text{ kipft}) + ((-2.376 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3664 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.3011 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(5.3011 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.301 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.301 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.92191$$

Status: PASS
Ratio: 0.920

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.418 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.27613 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.27613 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.13806$$

Status: PASS
Ratio: 0.140

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.37834 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 3.3664 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3664 \text{ kipft/ft}) \times (5.75 \text{ ft})) + \left(3 \times (-0.37834 \text{ kip/ft}) \times (5.75 \text{ ft})^2\right)}{(6 \times (3.3664 \text{ kipft/ft})) + (4 \times (-0.37834 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9776 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3664 \text{ kipft/ft})) + (3 \times (-0.37834 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3664 \text{ kipft/ft})) + (2 \times (-0.37834 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.19002 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.3664 \text{ kipft/ft})) + ((-0.37834 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.82704 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9776 \text{ ft})}{2}$$

$$p_a = 0.29832 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.19002 \text{ kip/ft}^2\right)}{\left(0.29832 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.63697$$

Status: **PASS**
Ratio: **0.640**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

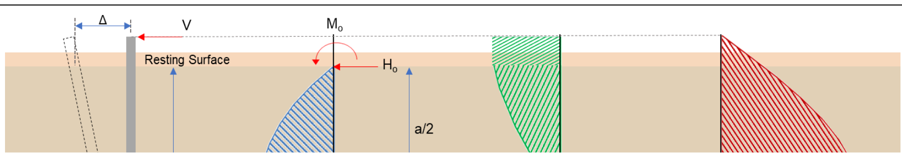
$Ratio$ - Lateral soil capacity

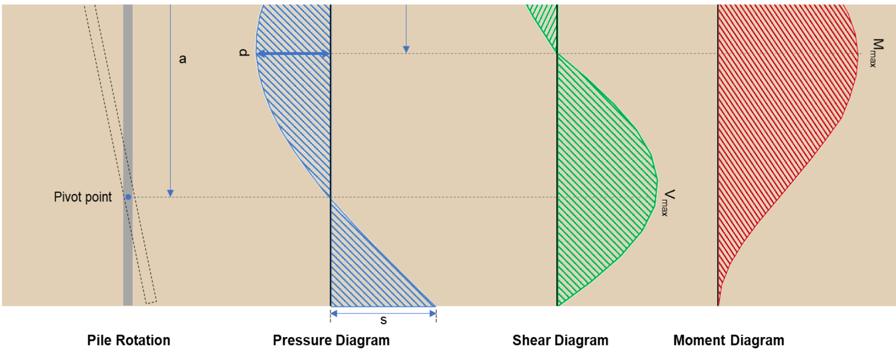
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.82704 \text{ kip/ft}^2\right)}{\left(0.8625 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.95888$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.96\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.63057\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.76\ \text{kipft}) + ((-3.96\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 5.6943\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6943\ \text{kipft/ft})}{(-0.63057\ \text{kip/ft})}$$

$$E = 9.0303\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6943\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.63057\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (5.6943\ \text{kipft/ft})) + (4 \times (-0.63057\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9761\ \text{ft}$$

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

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

$$V_{max} = ((-0.63057\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.0303\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9761\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (9.0303\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9761\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.6726\ \text{kip}$$

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

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

$$M_{max} = ((-0.63057\ \text{kip/ft}) \times (48\ \text{in}) \times (5.75\ \text{ft})) \times \left[\left(\frac{(9.0303\ \text{ft})}{(5.75\ \text{ft})} + \frac{(3.9761\ \text{ft})}{2 \times (5.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (9.0303\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9761\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (9.0303\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9761\ \text{ft})}{2 \times (5.75\ \text{ft})}\right)^4\right]\right]$$

	$$M_{max} = 23.619 \text{ kipft}$$		

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(6.252 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.388 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.388 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

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**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \mathrm{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(6.252\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$ $$Ratio = 0.002337$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\ \mathrm{in}$ - Effective width, d - Effective depth $$d = 0.80\ D$$ $$d = 0.80 \times (48\ \mathrm{in})$$ $$d = 38.4\ \mathrm{in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\ \mathrm{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\,\lambda_s\,\sqrt{f'_{ck}}\,b_w\,d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,max} = 296.21\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 6.252\ \mathrm{kip} \rightarrow 6252\ \mathrm{lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \frac{P}{6\,A_g}\right]\,b_w\,d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(6252\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,a} = 119.32\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \left(0.05\,f'_{ck}\right)\right]\,b_w\,d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + (0.05 \times (2500\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,b} = 348.89\ \mathrm{kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\ \mathrm{kip}), (119.32\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$ $$V_c = 119.32\ \mathrm{kip}$$	

	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) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{s,a} = 737.28\ \text{kip}$$	
	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) $$V_{s,b} = \frac{2\, A_v\, f_{ywk}\, d}{s_{ties}}$$ $$V_{s,b} = \frac{2 \times \left(0.11045\ \text{in}^2\right) \times (60\ \text{ksi}) \times (38.4\ \text{in})}{(10\ \text{in})}$$ $$V_{s,b} = 50.894\ \text{kip}$$	
	V_s - Governing shear strength of steel $$V_s = MIN\left[V_{s,a}, V_{s,b}\right]$$ $$V_s = MIN\left[(737.28\ \text{kip}), (50.894\ \text{kip})\right]$$ $$V_s = 50.894\ \text{kip}$$	
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((119.32\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 110.64\ \text{kip}$$	
	Considering x-direction: $V_{max} = 8.6726\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(8.6726\ \text{kip})}{(110.64\ \text{kip})}$$ $$Ratio = 0.078387$$	Status: **PASS** Ratio: **0.080**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$ $$S_m = 18432\ \text{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_c\, S_m$$	

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$

$$\phi M_{n,2} = 2121.6 \text{ kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

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

$$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 23.619 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(23.619 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.094626$$

Status: **PASS**
Ratio: **0.090**