Your Project Calculations

Project Name: Brad pole mount 10

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Brad%20pole%20mount%2010&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=PF0XsQKpb0kG1mZ6nbpvuxq3J7zRYRYNcBAuUcFO0ZINNEVSygxKNCgUdyz84KIv

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-HD-24-L-5Hx2W-87G6
Duty Classification:	HD
Module Width:	40.00 in
Module Length:	80.00in
Number of Rows:	5
Number of Columns:	2
Total Number of Modules:	10
Desired Tilt Angle:	30
Front Edge Clearance:	8
Total Array Height at Tilt:	16.39 ft
Total Frame Length:	11.50 ft
Frame Weight:	793 lbs
Array Dimensions N/S:	16.88 ft
Array Dimensions E/W:	13.50 ft
Rail Length:	202.50 in
Rail Spacing:	3.33 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	12.22 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: 6.50 ft
Foundation Volume:	3.852 y^3
Foundation Result:	PASSED
Mount Twist:	0.000003 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Sault Ste. Marie, MI 49783, USA
Wind Speed:	100 mph
Snow Load:	70 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.030790 ksf

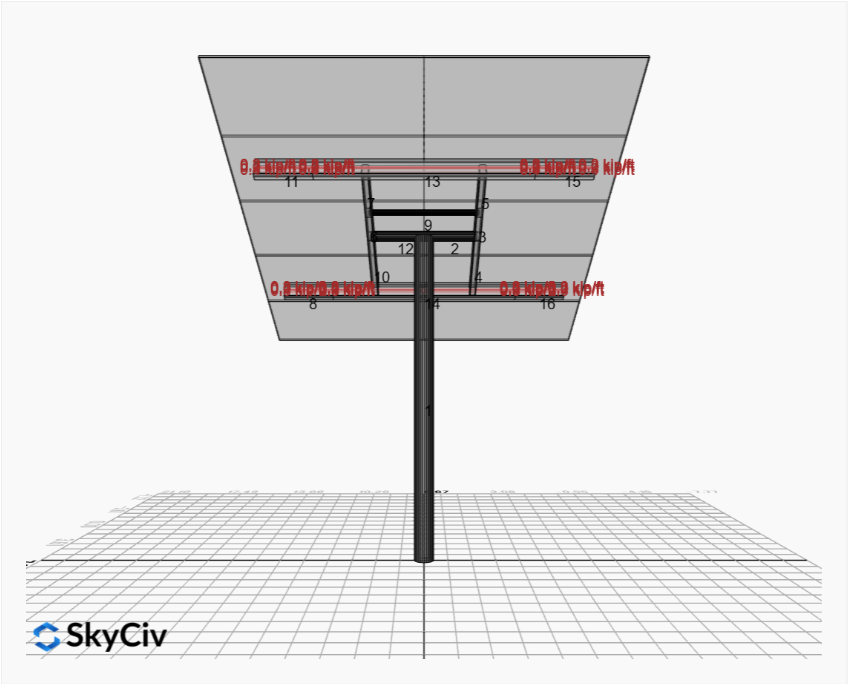
> **Design Disclaimer**
>
> This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

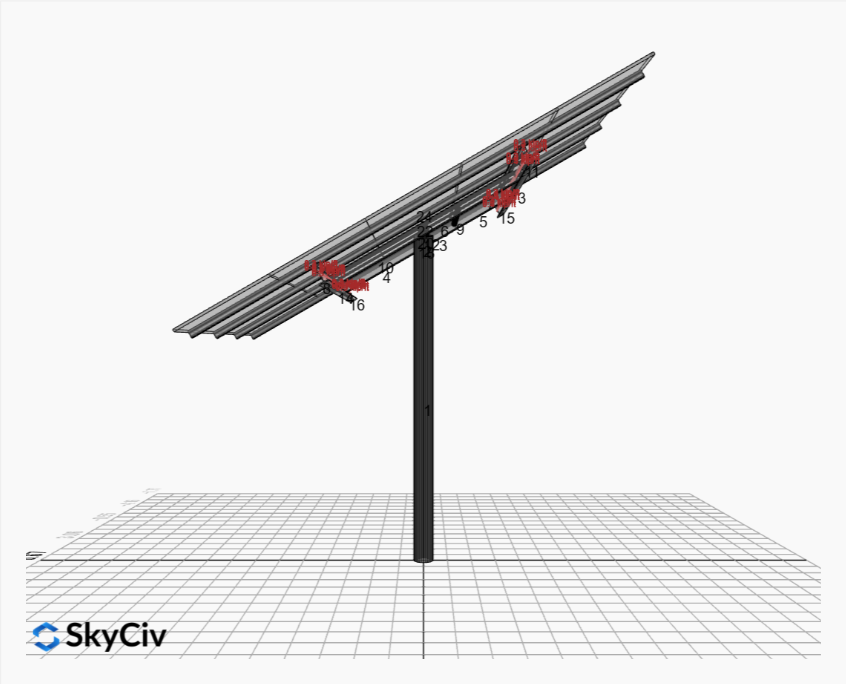
AutoDesigner Input

{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"product_type":"Beam","project_id":"Brad pole mount 10","site_address":"Sault Ste. Marie, MI 49783, USA","module_width":40,"module_length":80,"number_rows":"5","number_columns":2,"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":30,"ground_clearance":8,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}

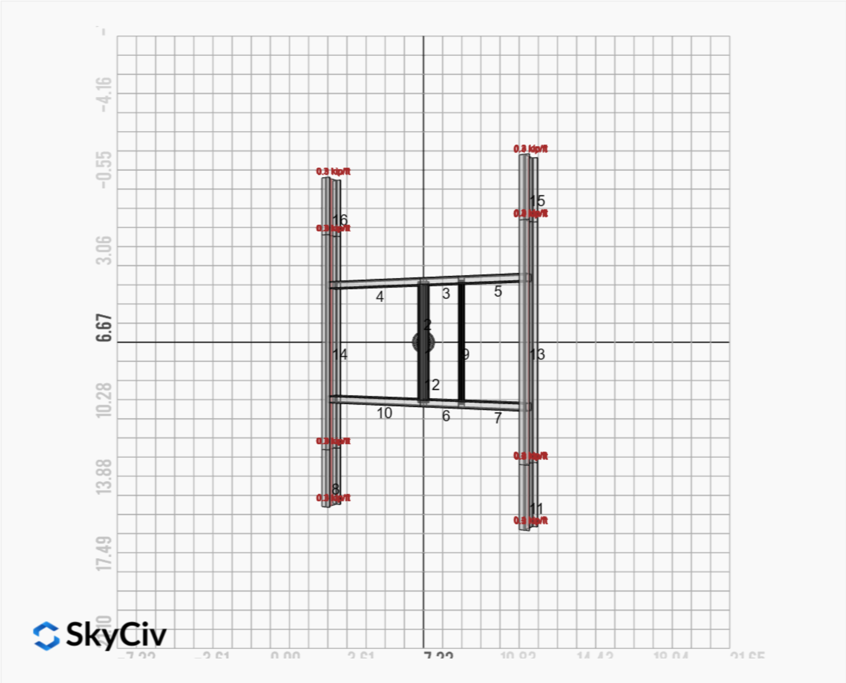
Design Notes:

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





SkyCiv



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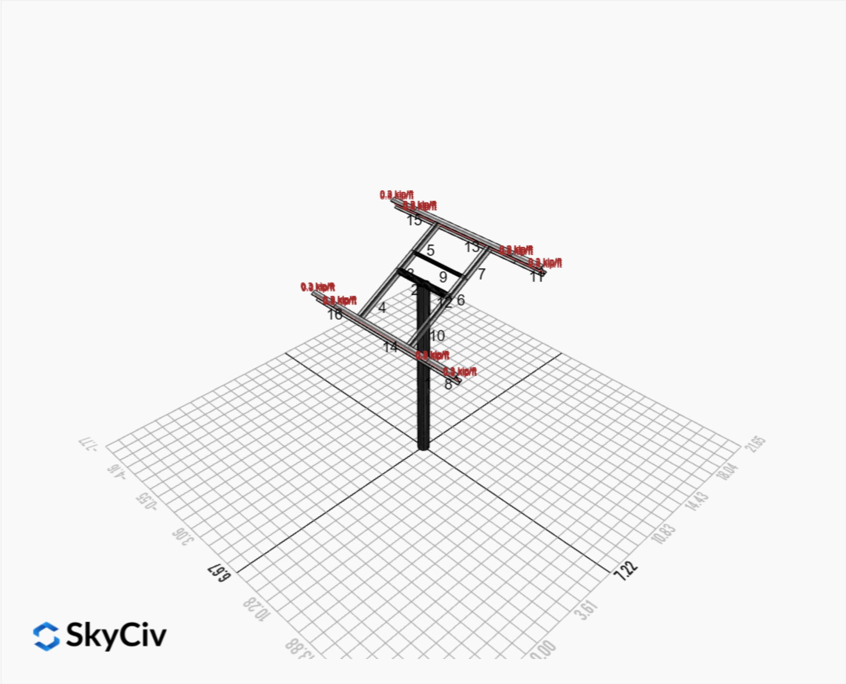
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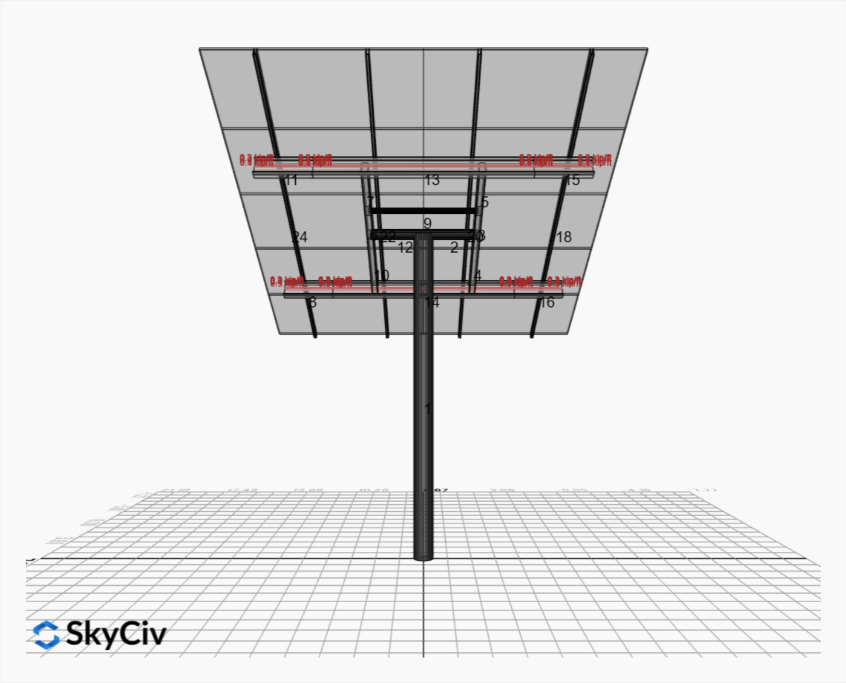
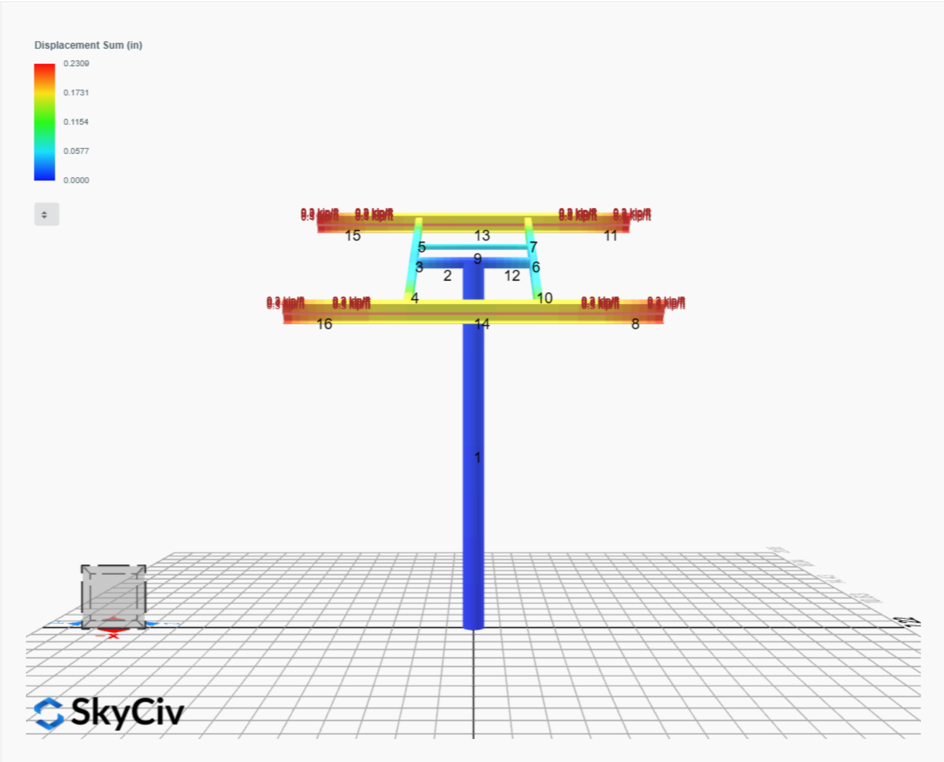
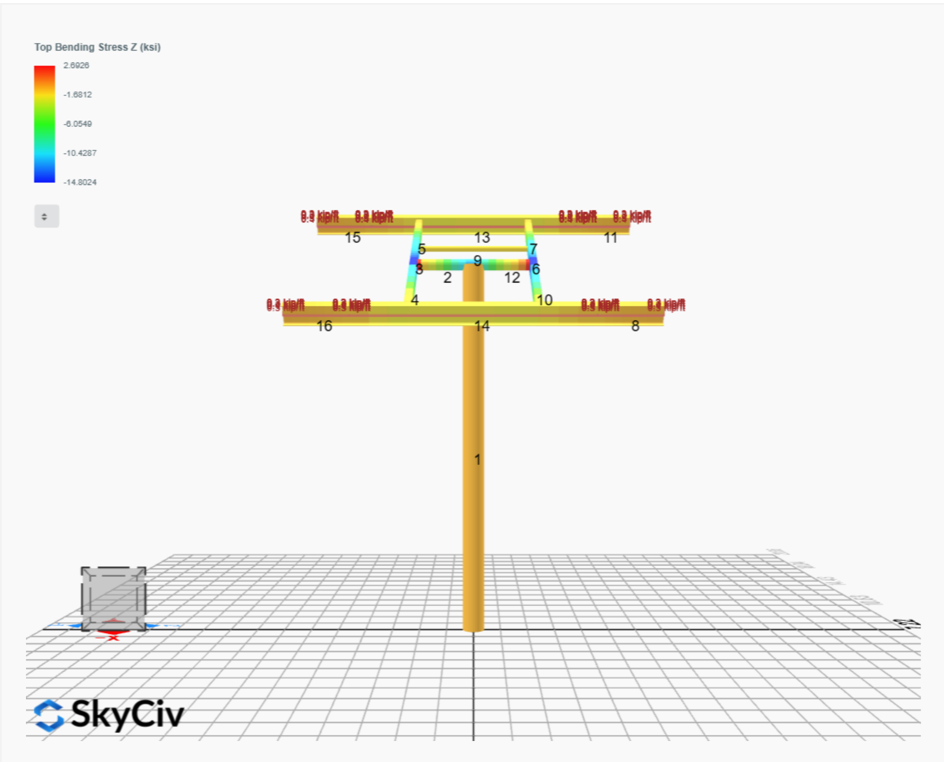
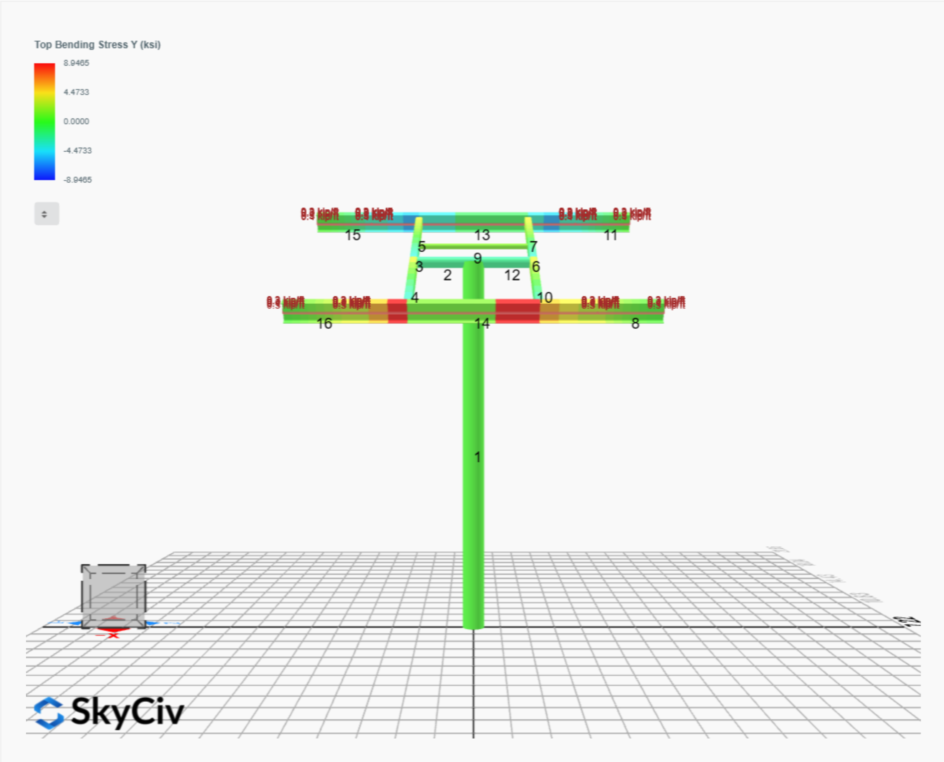
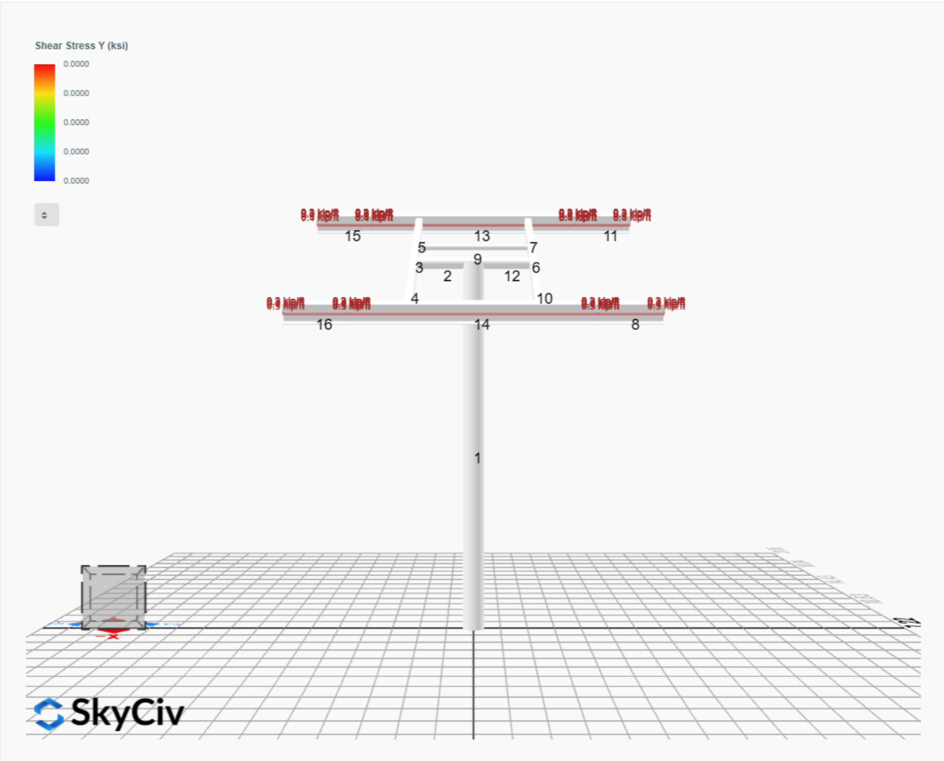

SkyCiv

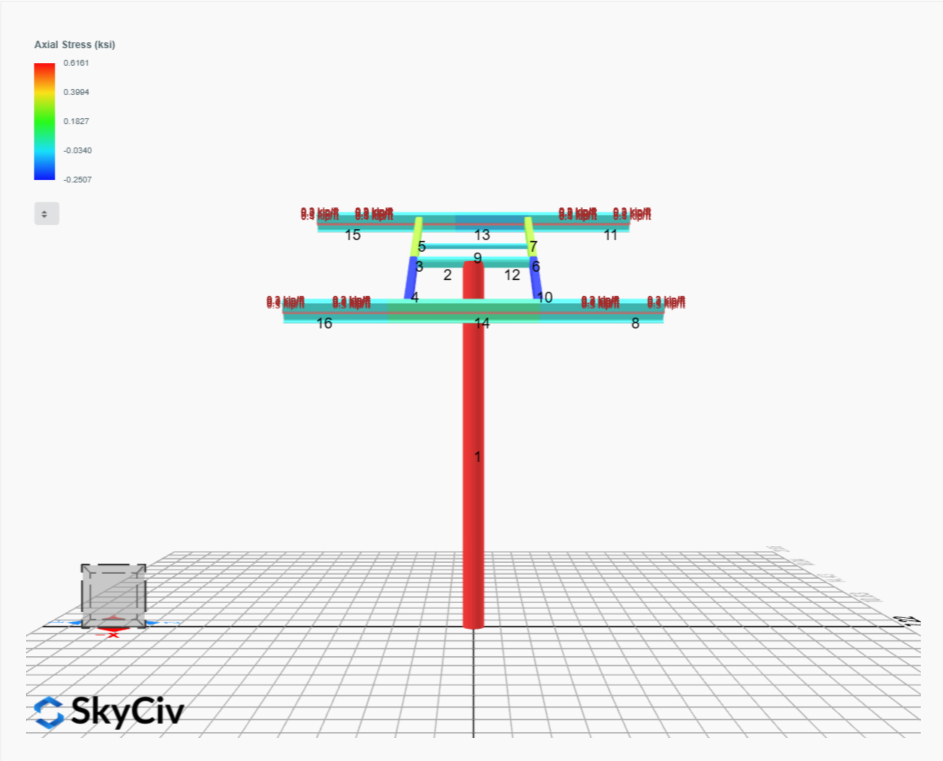
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
8.9465
4.4733
0.0000
-4.4733
-8.9465
0.3 kip/ft
15
13
11
5
7
3
9
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
0.0000
0.0000
0.0000
0.0000
0.0000
0.3 kip/ft
15
13
11
5
7
3
9
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.6161
0.3994
0.1827
-0.0340
-0.2507
15
13
11
5
7
3
9
6
2
12
4
10
16
14
8
1

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	1.8239	-0.0000	0.0000	-0.0000	0.0260
ULS: 2. D + L	0.0000	1.8239	-0.0000	0.0000	-0.0000	0.0260
ULS: 3. D + (S or Lr or R)	-0.0000	6.9985	-0.0000	0.0000	-0.0000	0.0565
ULS: 3. D + (S or Lr or R)	0.0000	1.8239	-0.0000	0.0000	-0.0000	0.0260
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	5.7049	-0.0000	0.0000	-0.0000	0.0489
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.8239	-0.0000	0.0000	-0.0000	0.0260
ULS: 5b. D + 0.7E	0.0000	1.8239	-0.0000	0.0000	-0.0000	0.0260
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	5.7049	-0.0000	0.0000	-0.0000	0.0489
ULS: 8. 0.6D + 0.7E	0.0000	1.0944	-0.0000	0.0000	-0.0000	0.0156
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2039	5.6411	0.0000	0.0000	-0.0000	27.5543
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.2039	5.6411	0.0000	0.0000	-0.0000	27.5543
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8890	-1.4480	-0.0000	0.0000	-0.0000	-22.6359
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.5742	-0.9027	-0.0000	0.0000	-0.0000	-26.6077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6529	8.5678	-0.0000	0.0000	-0.0000	20.6951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6529	8.5678	-0.0000	0.0000	-0.0000	20.6951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4168	3.2510	-0.0000	0.0000	-0.0000	-16.9476
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1806	3.6599	-0.0000	0.0000	-0.0000	-19.9264
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6529	4.6868	0.0000	0.0000	-0.0000	20.6722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.6529	4.6868	0.0000	0.0000	-0.0000	20.6722
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4168	-0.6300	-0.0000	0.0000	-0.0000	-16.9704
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1806	-0.2210	-0.0000	0.0000	-0.0000	-19.9493
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2039	4.9116	0.0000	0.0000	-0.0000	27.5439
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.2039	4.9116	0.0000	0.0000	-0.0000	27.5439
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8890	-2.1776	-0.0000	0.0000	-0.0000	-22.6463
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5742	-1.6322	-0.0000	0.0000	-0.0000	-26.6181

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	13.6491
Shear X	-3.6731
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	46.7605

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	8.5678
Shear X	-2.2039
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	27.5543

Project Details

Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	Brad pole mount 10
Unit System:	imperial

Design Input Information

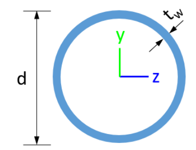
Design Factors

Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

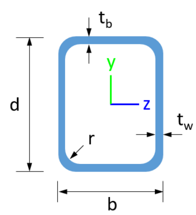
Design Materials

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

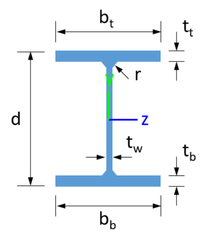
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					



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



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

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	25.66	25.66	12.22	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.59,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.74,1.67,1.67,1.66,1.63	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
8	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.59,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.74,1.67,1.67,1.66,1.63	300	200	1
11	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10	300	200	1
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.38,1.08,1.08,1.09,1.03,1.08,1.08,1.08,1.09,1.08,1.08,1.09,2.44,1.08,1.08,1.09,1.05	300	200	1
15	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	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	377.97	169.28	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	102.39	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94

10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	102.39	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.75	6.12	40.24	43.62
14	133.20	104.94	23.60	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62

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	0.081	0.561	0.000	0.032	0.000	0.594	#13	0.524	Not Required	Pass
2	0.000	0.499	0.169	0.111	0.031	0.623	#21	0.053	Not Required	Pass
3	0.011	0.732	0.088	0.073	0.026	0.826	#21	0.045	Not Required	Pass
4	0.011	0.726	0.092	0.073	0.022	0.813	#21	0.080	Not Required	Pass
5	0.011	0.455	0.091	0.073	0.021	0.473	#21	0.074	Not Required	Pass
6	0.011	0.732	0.088	0.073	0.026	0.826	#21	0.045	Not Required	Pass
7	0.011	0.455	0.091	0.073	0.021	0.473	#21	0.074	Not Required	Pass
8	0.000	0.032	0.070	0.026	0.010	0.102	#21	Not Required	Not Required	Pass
9	0.002	0.059	0.042	0.001	0.000	0.102	#21	0.136	Not Required	Pass
10	0.011	0.726	0.092	0.073	0.022	0.813	#21	0.080	Not Required	Pass
11	0.000	0.032	0.070	0.026	0.010	0.102	#21	Not Required	Not Required	Pass
12	0.000	0.499	0.169	0.111	0.031	0.623	#21	0.053	Not Required	Pass
13	0.004	0.171	0.245	0.049	0.018	0.393	#21	0.190	Not Required	Pass
14	0.005	0.177	0.245	0.049	0.018	0.395	#21	0.190	Not Required	Pass
15	0.000	0.032	0.070	0.026	0.010	0.102	#21	Not Required	Not Required	Pass
16	0.000	0.032	0.070	0.026	0.010	0.102	#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

OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

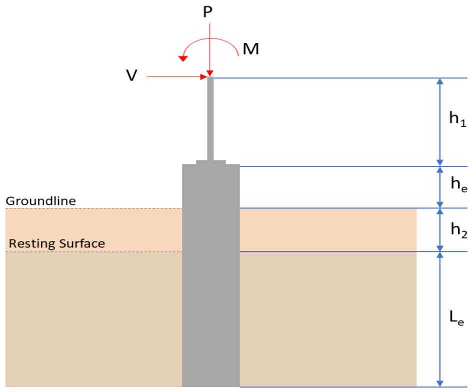
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 6.5$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ 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)	8.568	13.649
V_x (kip)	-2.204	-3.673
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	27.554	46.761

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.204 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.35096 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.554\ \mathrm{kipft}) + ((-2.204\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.3876\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.0665\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.0665\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 6.066\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.5\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.066\ \mathrm{ft})}{(6.5\ \mathrm{ft})}$$

$$Ratio = 0.93323$$

Status: **PASS**
Ratio: **0.930**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.568\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.5355\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.5355\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.26775$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.35096\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.3876\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3876\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.35096\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (4.3876\ \mathrm{kipft/ft})) + (4 \times (-0.35096\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4728\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.3876\ \mathrm{kipft/ft})) + (3 \times (-0.35096\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^2}{(6.5\ \mathrm{ft})^2 \times [(3 \times (4.3876\ \mathrm{kipft/ft})) + (2 \times (-0.35096\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$

$$p = 0.23661\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.3876\ \mathrm{kipft/ft})) + ((-0.35096\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$

$$s = 0.92222\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4728\ \mathrm{ft})}{2}$$

$$p_a = 0.33546\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.23661\ \mathrm{kip/ft^2}\right)}{\left(0.33546\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.70533$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.5\ \mathrm{ft})$$

$$p_s = 0.975\ \mathrm{kip/ft^2}$$

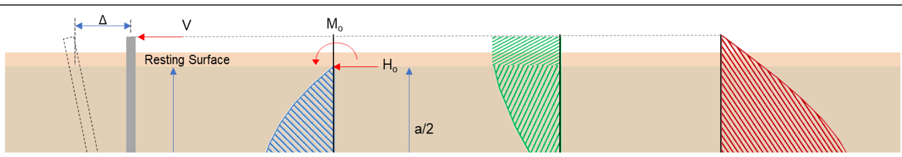
$Ratio$ - Lateral soil capacity

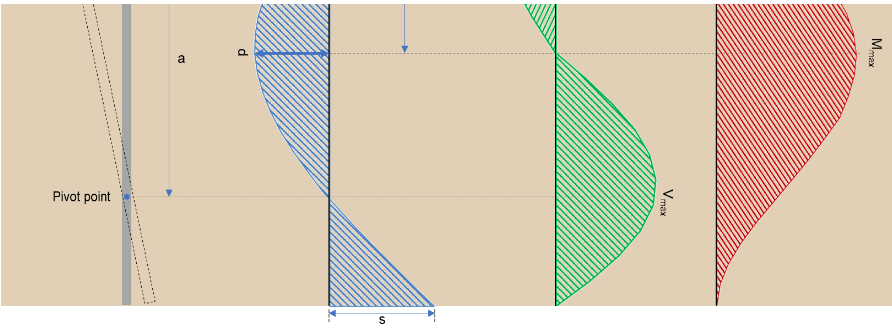
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.92222\ \mathrm{kip/ft^2}\right)}{\left(0.975\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.94586$$

Status: **PASS**
Ratio: **0.950**





Pile Rotation   Pressure Diagram   Shear Diagram   Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.673\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.58487\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.761\ \mathrm{kipft}) + ((-3.673\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 7.446\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.446\ \mathrm{kipft/ft})}{(-0.58487\ \mathrm{kip/ft})}$$

$$E = 12.731\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.446\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.58487\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (7.446\ \mathrm{kipft/ft})) + (4 \times (-0.58487\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4709\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.58487\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.731\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.4709\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (12.731\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.4709\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.6524\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.58487\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.5\ \mathrm{ft})) \times \left[\left(\frac{(12.731\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + \frac{(4.4709\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (12.731\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.4709\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (12.731\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.4709\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 29.988 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(13.649 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.143 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.143 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

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,

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

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

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

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,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$

$$\phi P_N = 2675.2\ \mathrm{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(13.649\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0051021$$

Status: **PASS**
Ratio: **0.010**

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,max} = 296.21\ \mathrm{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 13.649\ \mathrm{kip} \rightarrow 13649\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(13649\ \mathrm{lbf})}{6 \times \left(2304\ \mathrm{in}^2\right)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 120.31\ \mathrm{kip}$$

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

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

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + (0.05 \times (2500\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,b} = 348.89\ \mathrm{kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120.31\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120.31\ \mathrm{kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.31\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 111.28\ \text{kip}$$

Considering x-direction:

$V_{max} = 9.6524\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.6524\ \text{kip})}{(111.28\ \text{kip})}$$

$$Ratio = 0.086741$$

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

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:
$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$

$$\phi M_{n,1} = 249.600 kipft$$

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

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

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

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

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

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:

$M_{max} = 29.988\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(29.988\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.12014$$

Status: **PASS**
Ratio: **0.120**