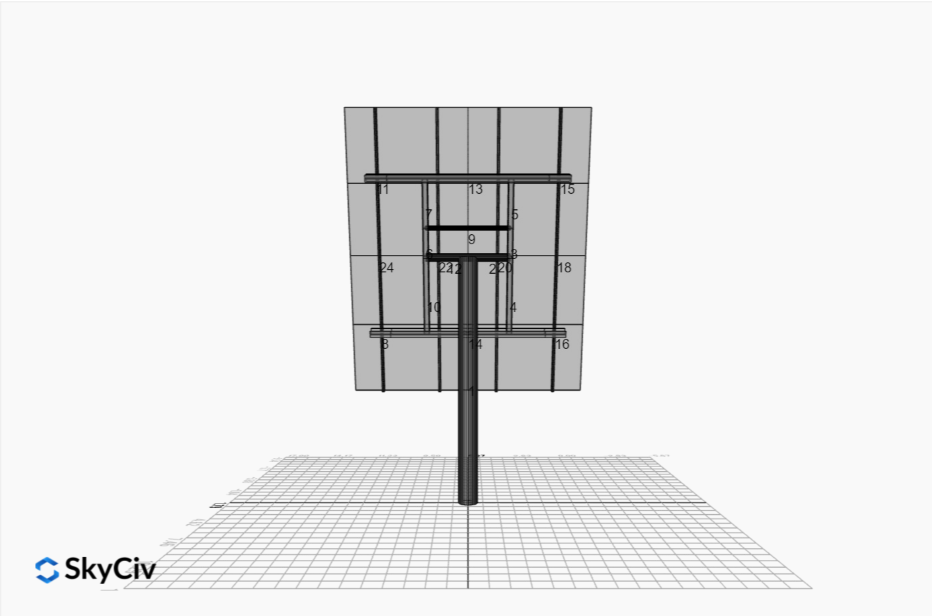
Project Details

Project Name: Wanoka Campground
Location: Iron River, WI 54847, USA
Unique ID: 1P-0-10TOP-SD-12-L-4Hx2W-1L1J
Dealer: ______________________________

Date: Mon Oct 07 2024
Number of Modules: 8
Number of Poles: 1
Date Sold: ______________________________



Array Dimensions N/S	13.83 ft
Array Dimensions E/W	11.33 ft
Winter Tilt Angle	75
Front Edge Clearance	5 ft

MT Solar Bill of Materials (1P-0-10TOP-SD-12-L-4Hx2W-1L1J)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-12	12IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	2

Rail Bill of Materials

Part	Qty
Rails (164in)	4
Rail Attachment	8
Module Mid Clamp	12
Module End Clamp	8
Ground Lug	2

Site Details:

Site Address: Iron River, WI 54847, USA

Array Specification

Duty Classification:	SD
Module Width:	41.00 in
Module Length:	67.00in
Number of Rows:	4
Number of Columns:	2
Total Number of Modules:	8
Winter Tilt Angle:	75
Front Edge Clearance:	5
Total Array Height at Tilt:	18.36 ft
Total Frame Length:	9.50 ft
Frame Weight:	951 lbs
Array Dimensions N/S:	13.83 ft
Array Dimensions E/W:	11.33 ft
Rail Length:	166.00 in
Rail Spacing:	2.83 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	11.68 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

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Iron River, WI 54847, USA
Wind Speed:	99 mph
Snow Load:	60 psf

Design Disclaimer

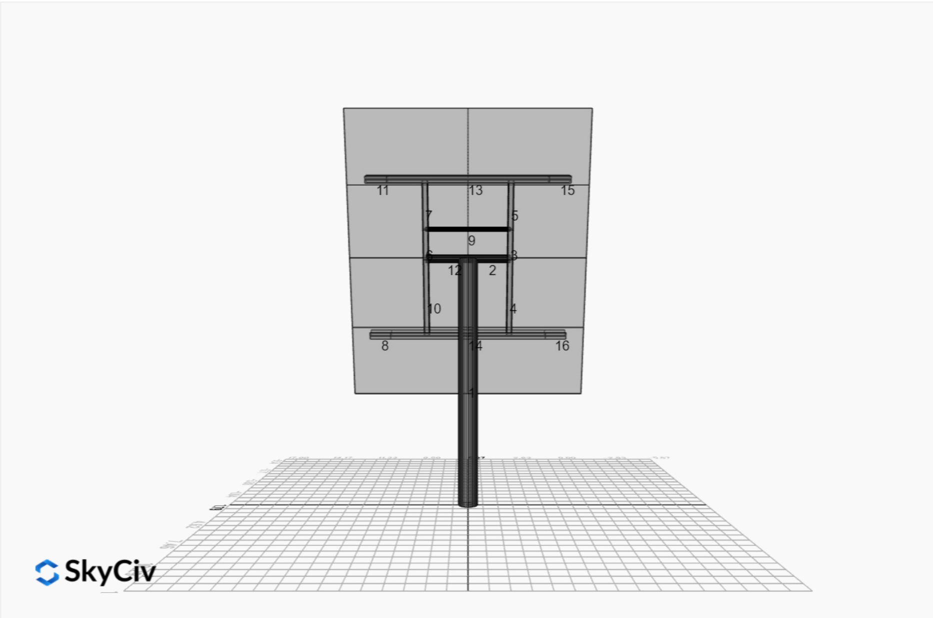
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

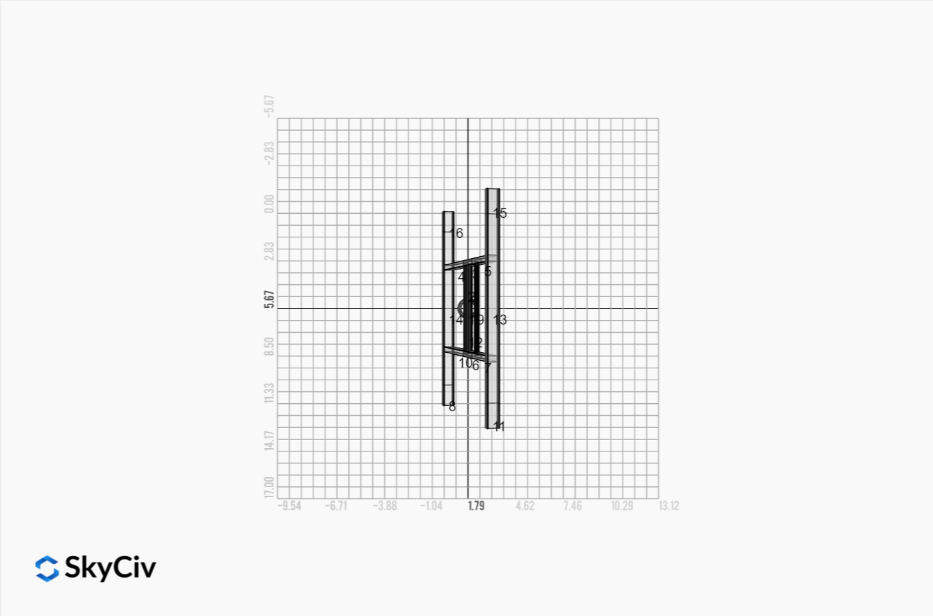
AutoDesigner Input

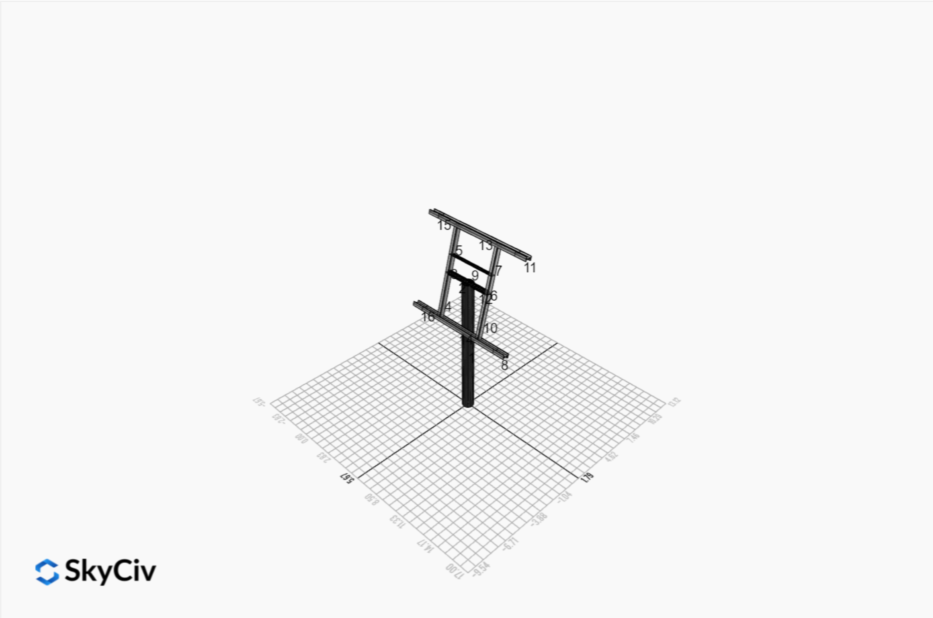
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"Tyler Stephenson","designer_email":"tylers@jolmaelectric.com","designer_phone":"732-597-6657","project_id":"Wanoka Campground","site_address":"Iron River, WI 54847, USA","module_width":41,"module_length":67,"number_rows":"4","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":75,"ground_clearance":5,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"10_40","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails":false}

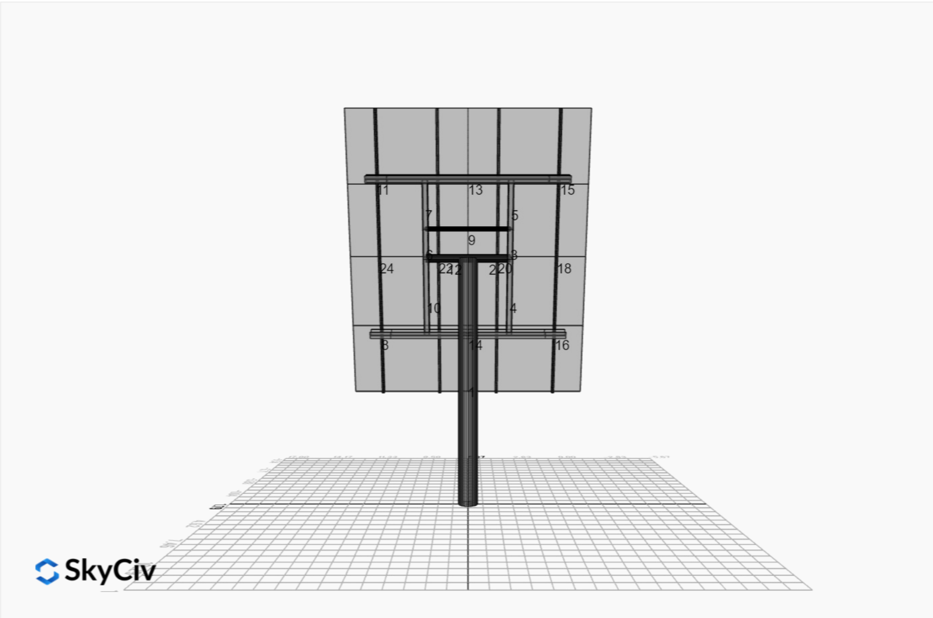
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation 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

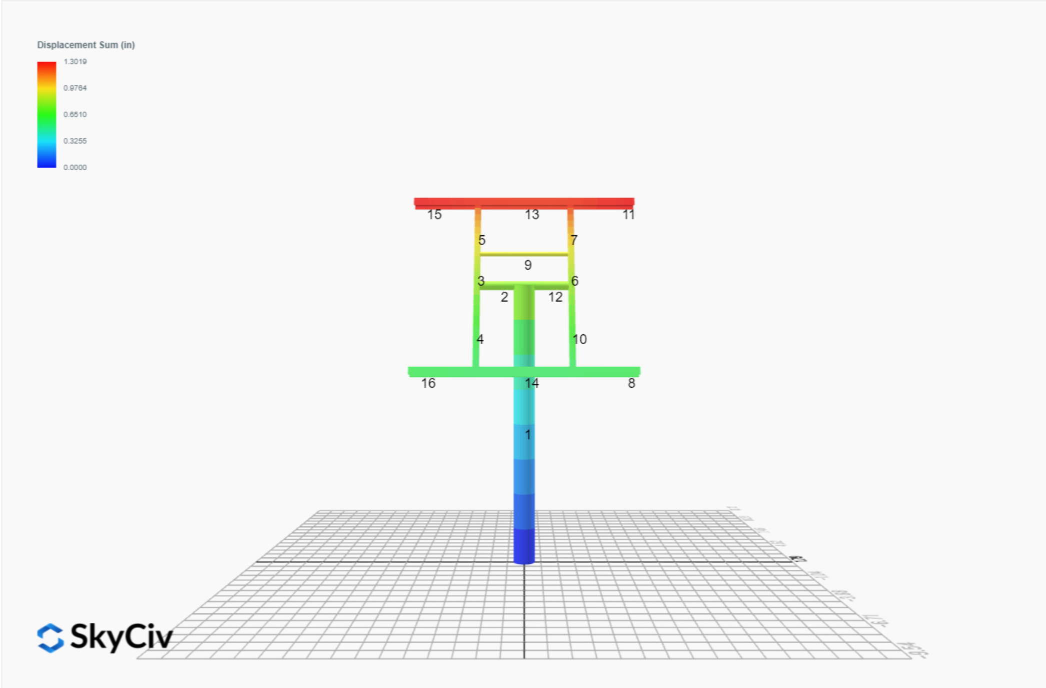


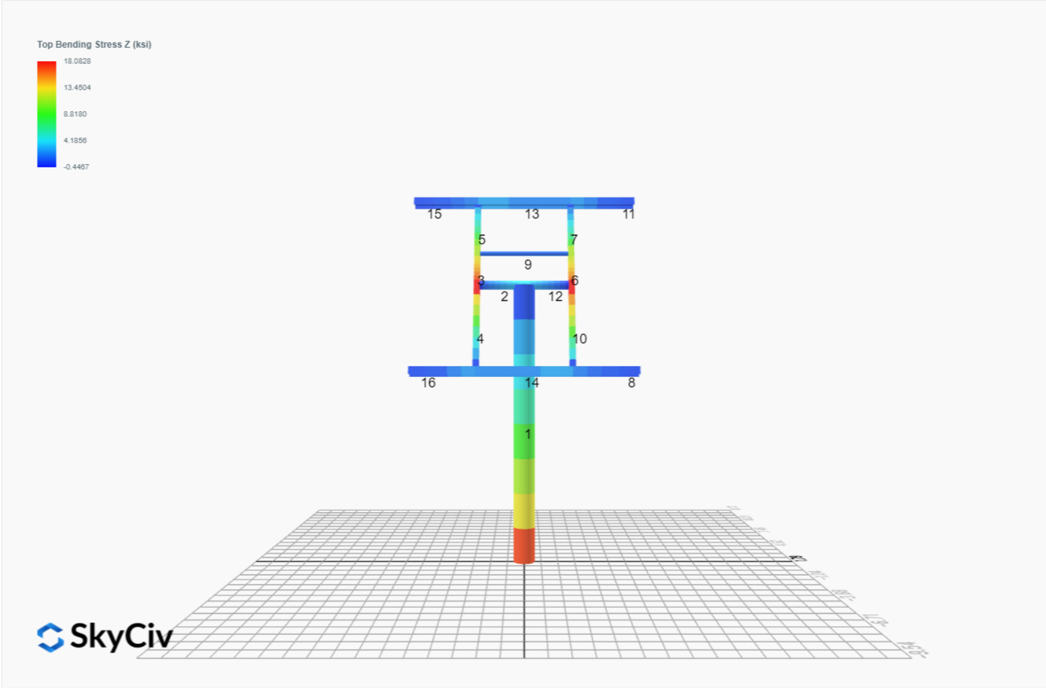


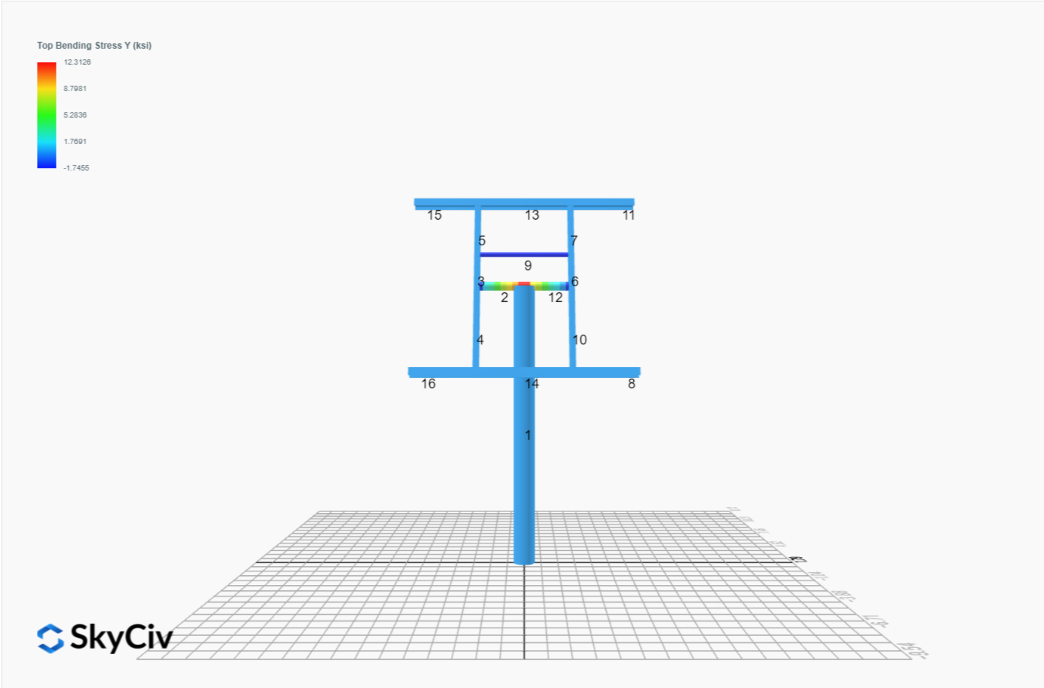


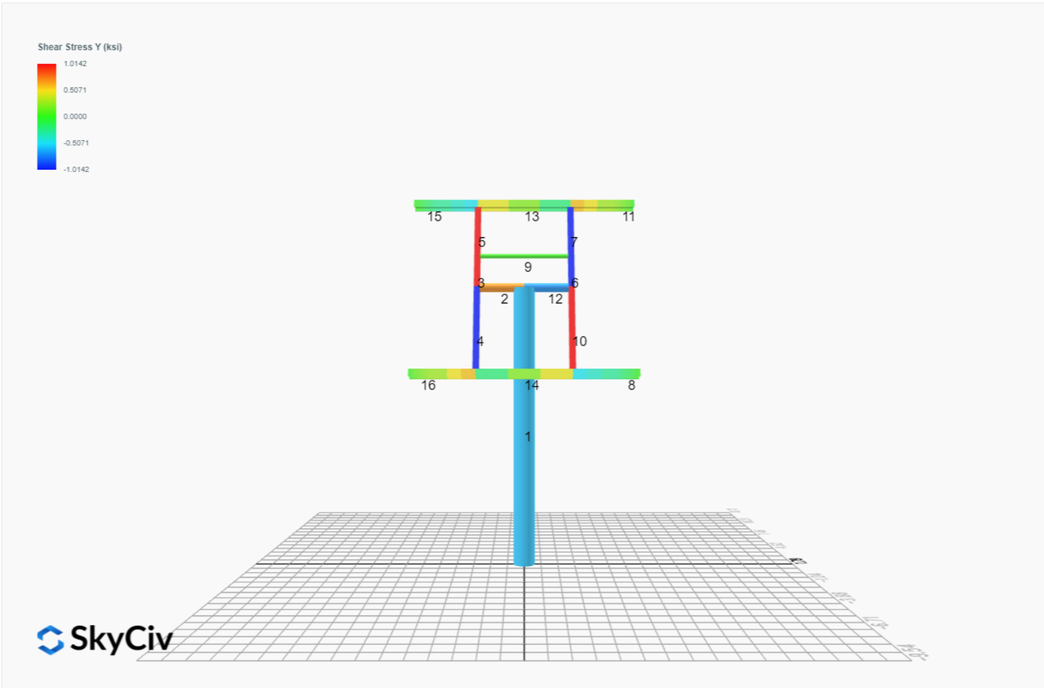


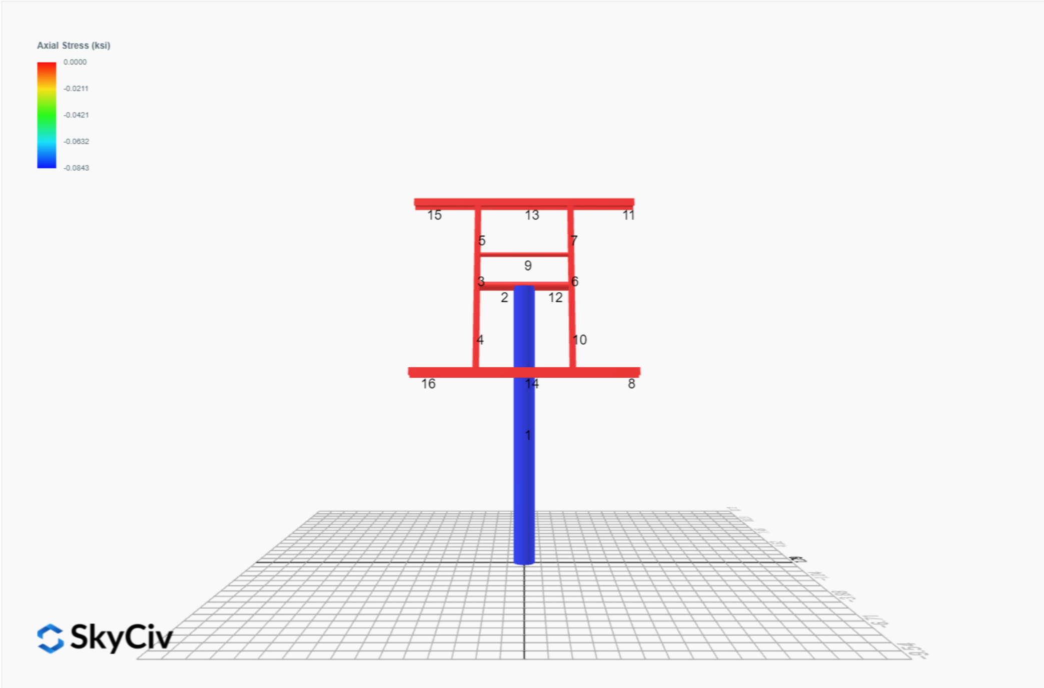
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
12.3126
8.7981
5.2836
1.7691
-1.7455
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
1.0142
0.5071
0.0000
-0.5071
-1.0142
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.0000
-0.0211
-0.0421
-0.0632
-0.0843
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	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 2. D + L	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 3. D + (S or Lr or R)	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 3. D + (S or Lr or R)	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 5b. D + 0.7E	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 8. 0.6D + 0.7E	0.0000	0.9251	0.0000	-0.0000	0.0000	0.0036
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2469	2.1439	0.0000	-0.0000	0.0000	26.2891
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2469	0.9398	0.0000	-0.0000	0.0000	-26.2032
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6852	1.9934	0.0000	-0.0000	0.0000	19.7183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6852	1.0903	0.0000	-0.0000	0.0000	-19.6509
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6852	1.9934	0.0000	-0.0000	0.0000	19.7183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6852	1.0903	0.0000	-0.0000	0.0000	-19.6509
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.5419	0.0000	-0.0000	0.0000	0.0059
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2469	1.5272	0.0000	-0.0000	0.0000	26.2867
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.9251	0.0000	-0.0000	0.0000	0.0036
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2469	0.3231	0.0000	-0.0000	0.0000	-26.2056
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.9251	0.0000	-0.0000	0.0000	0.0036

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	2.8537
Shear X	-3.7448
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	43.9434

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	2.1439
Shear X	-2.2469
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	26.2891

Project Details

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

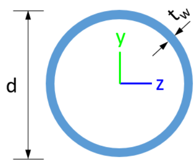
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

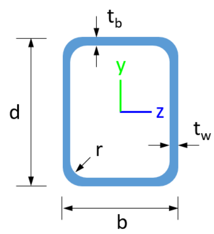
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

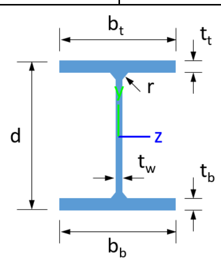
Section Dimensions



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					
11	10in Pipe Sch 40	10.75	0.36					



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
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38
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	11	24.53	24.53	11.68	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
4	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
8	18	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
11	18	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	300	200	1
14	18	4.88	4.00	7.50	1.15,1.15,1.15,1.15,1.15,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15,1.16,1.15	300	200	1
15	18	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	2.10	2.10	1.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1

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	535.87	335.13	147.68	147.68	160.76	160.76
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	113.97	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	6.26	29.14	16.61
11	120.60	113.97	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	20.41	6.45	30.09	45.74
14	120.60	84.03	20.28	6.45	30.09	45.74

15	120.60	113.97	23.36	6.45	30.09	45.74
16	120.60	113.97	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	0.009	0.298	0.000	0.023	0.000	0.302	#13	0.401	Not Required	Pass
2	0.001	0.106	0.209	0.027	0.044	0.315	#13	0.052	Not Required	Pass
3	0.005	0.358	0.050	0.036	0.014	0.403	#13	0.044	Not Required	Pass
4	0.004	0.357	0.030	0.036	0.005	0.385	#13	0.078	Not Required	Pass
5	0.004	0.222	0.019	0.036	0.003	0.226	#13	0.073	Not Required	Pass
6	0.005	0.358	0.050	0.036	0.014	0.403	#13	0.044	Not Required	Pass
7	0.004	0.222	0.019	0.036	0.003	0.226	#13	0.073	Not Required	Pass
8	0.000	0.005	0.005	0.007	0.001	0.009	#13	Not Required	Not Required	Pass
9	0.004	0.010	0.031	0.001	0.000	0.041	#13	0.132	Not Required	Pass
10	0.004	0.357	0.030	0.036	0.005	0.385	#13	0.078	Not Required	Pass
11	0.000	0.005	0.005	0.007	0.001	0.009	#13	Not Required	Not Required	Pass
12	0.001	0.106	0.209	0.027	0.044	0.315	#13	0.052	Not Required	Pass
13	0.000	0.050	0.038	0.020	0.004	0.073	#13	0.177	Not Required	Pass
14	0.001	0.052	0.038	0.020	0.004	0.074	#13	0.177	Not Required	Pass
15	0.000	0.005	0.005	0.007	0.001	0.009	#13	Not Required	Not Required	Pass
16	0.000	0.005	0.005	0.007	0.001	0.009	#13	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

REFERENCES	CALCULATIONS	RESULTS

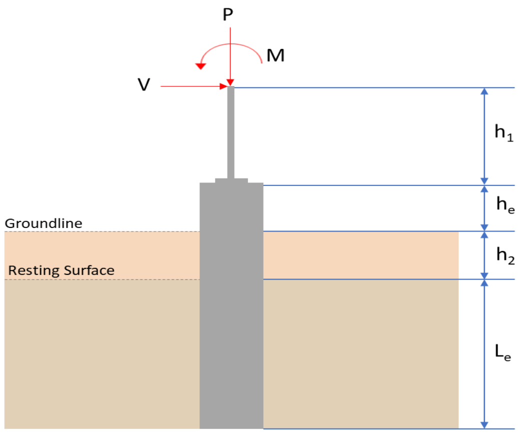
SkyCiv Foundation Design

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
$L = 6.5\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	2.144	2.854
V_x (kip)	-2.247	-3.745
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	26.289	43.943

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,

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

$$H_o = \frac{(-2.247\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.3578\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.289\ \mathrm{kipft}) + ((-2.247\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.1861\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 5.9226\ \mathrm{ft}$ - 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,

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

$$L_{e,req} = MAX[(5.9226\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 5.923\ \mathrm{ft}$$

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

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

$$Ratio = \frac{(5.923\ \mathrm{ft})}{(6.5\ \mathrm{ft})}$$

$$Ratio = 0.91123$$

Status: **PASS**
Ratio: **0.910**

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

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

$$q = \frac{(2.144\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.134\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.134\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.067$$

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

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:

$H_o = -0.3578\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.1861\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1861\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.3578\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (4.1861\ \mathrm{kipft/ft})) + (4 \times (-0.3578\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4797\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1861\ \mathrm{kipft/ft})) + (3 \times (-0.3578\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^{\ 2}}{(6.5\ \mathrm{ft})^2 \times [(3 \times (4.1861\ \mathrm{kipft/ft})) + (2 \times (-0.3578\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$

$$p = 0.21418\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.1861\ \mathrm{kipft/ft})) + ((-0.3578\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$

$$s = 0.85869\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.4797\ \mathrm{ft})}{2}$$

$$p_a = 0.33598\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.21418\ \mathrm{kip/ft^2}\right)}{\left(0.33598\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.63749$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

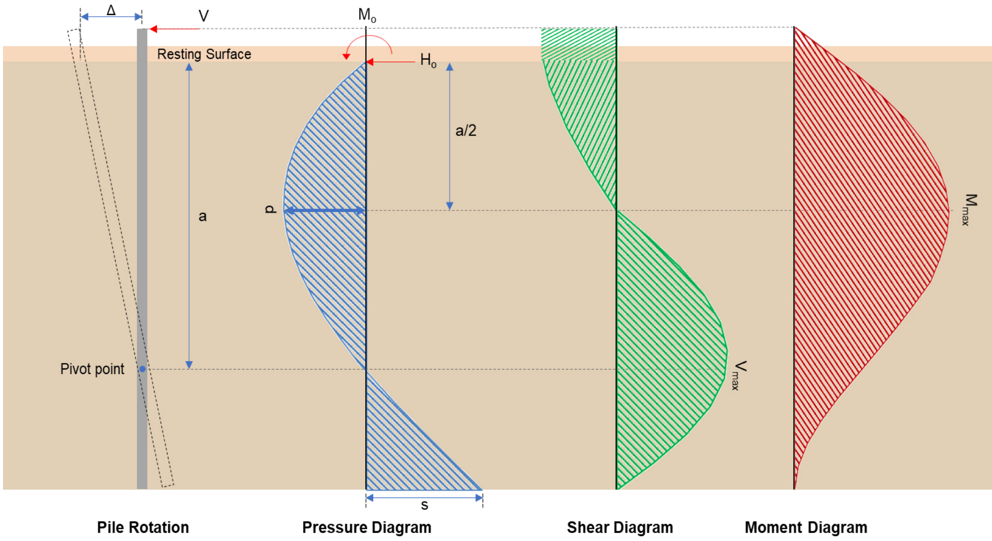
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.85869\ \mathrm{kip/ft^2}\right)}{\left(0.975\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.8807$$

Status: **PASS**
Ratio: **0.880**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.745\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.59634\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.943\ \mathrm{kipft}) + ((-3.745\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.9973\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.9973\ \mathrm{kipft/ft})}{(-0.59634\ \mathrm{kip/ft})}$$

$$E = 11.734\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.9973\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.59634\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (6.9973\ \mathrm{kipft/ft})) + (4 \times (-0.59634\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4794\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.59634\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.734\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.4794\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^2\right.\right.$$
$$\left.\left. + \left[4 \times \left(\frac{3 \times (11.734\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.4794\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^3\right]\right]\right]$$

$$V_{max} = 0.1032\ \mathrm{kip}$$

$V_{max} = 9.1932$ kip

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

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

$$M_{max} = ((-0.59634\text{ kip/ft}) \times (48\text{ in}) \times (6.5\text{ ft})) \times \left[\left(\frac{(11.734\text{ ft})}{(6.5\text{ ft})} + \frac{(4.4794\text{ ft})}{2 \times (6.5\text{ ft})}\right) - \left[\left(\frac{4 \times (11.734\text{ ft})}{(6.5\text{ ft})} + 3\right) \times \left(\frac{(4.4794\text{ ft})}{2 \times (6.5\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.734\text{ ft})}{(6.5\text{ ft})} + 2\right) \times \left(\frac{(4.4794\text{ ft})}{2 \times (6.5\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 28.469\text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5$ ksi - Concrete strength,
$f_{yk} = 60$ 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(2.854\text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\text{ ksi}) \times (2304\text{ in}^2))}{(60\text{ ksi}) - (0.85 \times (2.5\text{ ksi}))}, (0.08 \times (2304\text{ in}^2))\right]$$

$$A_{st,required} = -84.501\text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.501\text{ in}^2), (0.0018 \times (2304\text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

$$Ratio = \frac{(4.1472\text{ in}^2)}{(4.2951\text{ in}^2)}$$

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

	$$s_{rebar} = Max\left[1.5, (1.5 \times (0.625\ \text{in}))\right]$$ $$s_{rebar} = 1.5\ \text{in}$$		
	Ties:		
25.7.2.2	Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)		
25.7.2.1	s_{ties} - Maximum spacing of ties, $$s_{ties} = Min\left[(16\ d_{bar}), (48\ d_{ties}), Min\,(D, b)\right]$$ $$s_{ties} = Min\left[(16 \times (0.625\ \text{in})), (48 \times (0.375\ \text{in})), Min\,((48\ \text{in}), (48\ \text{in}))\right]$$ $$s_{ties} = 10\ \text{in}$$		
	Summary: Main reinforcement: **14 - #5 (0.625 in)** Ties: **#3(0.375 in) - 10 in**		
	Axial Compression Strength (ACI 318-19, LRFD)		
22.4.2.2	ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\ 0.80\ \left[\left(0.85\ f'_{ck}\ [A_g - A_{st}]\right) + (f_{yk}\ A_{st})\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \text{ksi}) \times \left[\left(2304\ \text{in}^2\right) - \left(4.2951\ \text{in}^2\right)\right]\right) + \left((60\ \text{ksi}) \times \left(4.2951\ \text{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \text{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(2.854\ \text{kip})}{(2675.2\ \text{kip})}$$ $$Ratio = 0.0010668$$		Status: **PASS** Ratio: **0.000**
	Shear Strength (ACI 318-19, LRFD)		
	Parameters:		
	$b_w = 48\ \text{in}$ - Effective width,		
22.5.2.2	d - Effective depth $$d = 0.80\ D$$ $$d = 0.80 \times (48\ \text{in})$$ $$d = 38.4\ \text{in}$$		
22.5.5.1.3	λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \text{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,		
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\ \lambda_s\ \sqrt{f'_{ck}}\ b_w\ d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \text{psi})} \times (48\ \text{in}) \times (38.4\ \text{in})$$ $$V_{c,max} = 296.21\ \text{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 2.854\ \text{kip} \rightarrow 2854\ \text{lbf}$,		
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\ \lambda_s\ \sqrt{f'_{ck}} + \frac{P}{6\ A_g}\right]\ b_w\ d$$		

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(2854\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 118.87\ \mathrm{kip}$$

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

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (118.87\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 118.87\ \mathrm{kip}$$

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

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

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

$$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{s,a} = 737.28\ \mathrm{kip}$$

A_v - Ties rebar area,

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

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

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

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

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

$$V_{s,b} = \frac{2 \times \left(0.11045\ \mathrm{in}^2\right) \times (60\ \mathrm{ksi}) \times (38.4\ \mathrm{in})}{(10\ \mathrm{in})}$$

$$V_{s,b} = 50.894\ \mathrm{kip}$$

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

$$V_s = 50.894\ \mathrm{kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((118.87\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 110.34\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 9.1932\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.1932\ \mathrm{kip})}{(110.34\ \mathrm{kip})}$$

$$Ratio = 0.083314$$

Status: **PASS**
Ratio: **0.080**

	Flexural Strength (ACI 318-19, LRFD)	

S_m - Section modulus

$$S_m = \frac{b\,D^2}{6}$$

$$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$

$$S_m = 18432\ \mathrm{in}^3$$

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

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

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

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

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

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 28.469\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(28.469\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$

$$Ratio = 0.11406$$

Status: **PASS**
Ratio: **0.110**