Your Project Calculations

Project Name: Stepman_JB_RevA

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Stepman_JB_RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/2_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=pNdHoL5MrpZT0d4nGKMn8Q3UogvqJlgwg3LOYlu28RmrnutURPrSgaAFnRcq2dy6

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-SD-45-L-4Hx3W-I210
Duty Classification:	SD
Module Width:	41.20 in
Module Length:	61.30in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	25
Front Edge Clearance:	6
Total Array Height at Tilt:	11.84 ft
Total Frame Length:	15.00 ft
Frame Weight:	677 lbs
Array Dimensions N/S:	13.90 ft
Array Dimensions E/W:	15.57 ft
Rail Length:	166.80 in
Rail Spacing:	2.55 ft
Rail Check:	

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.94 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.25 ft
Foundation Volume:	2.160 y^3
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	16891 Lively Wood Ln, Rough and Ready, CA 95975, USA
Wind Speed:	110 mph
Snow Load:	29 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.013006 ksf

Design Disclaimer

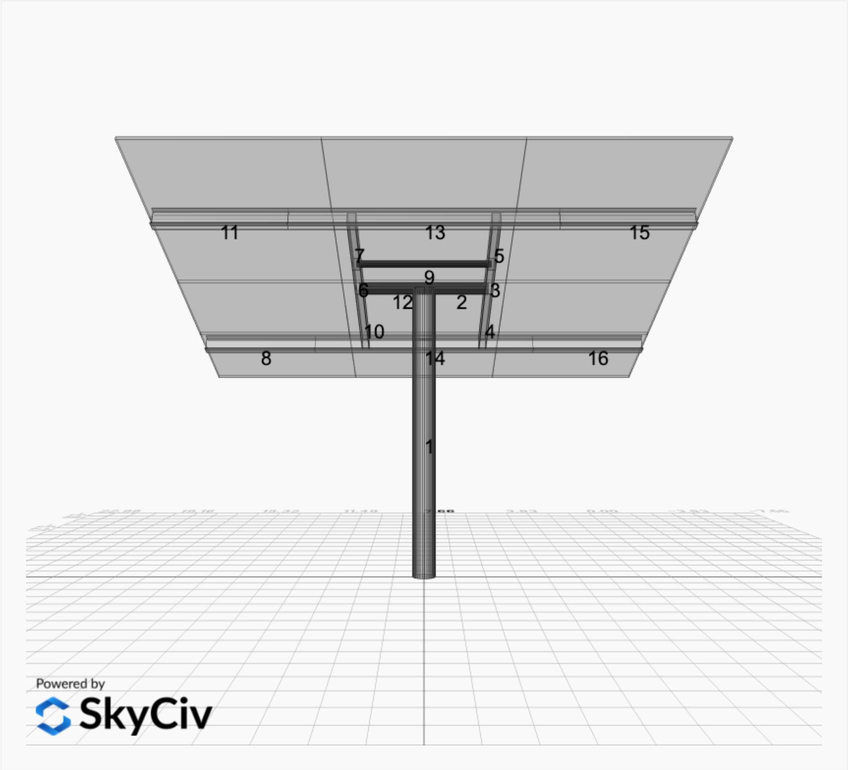
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

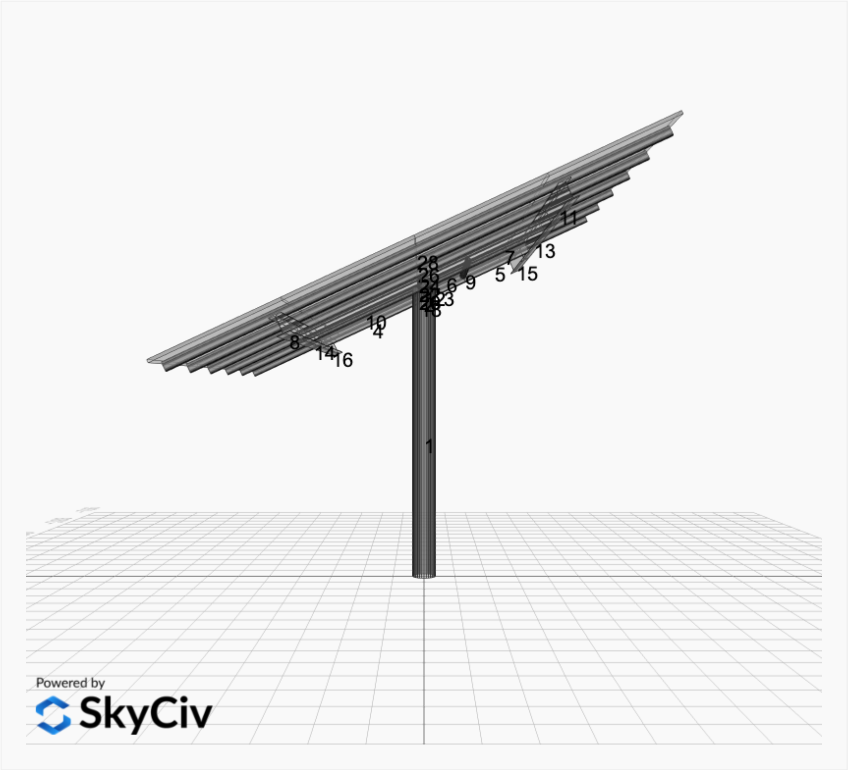
AutoDesigner Input

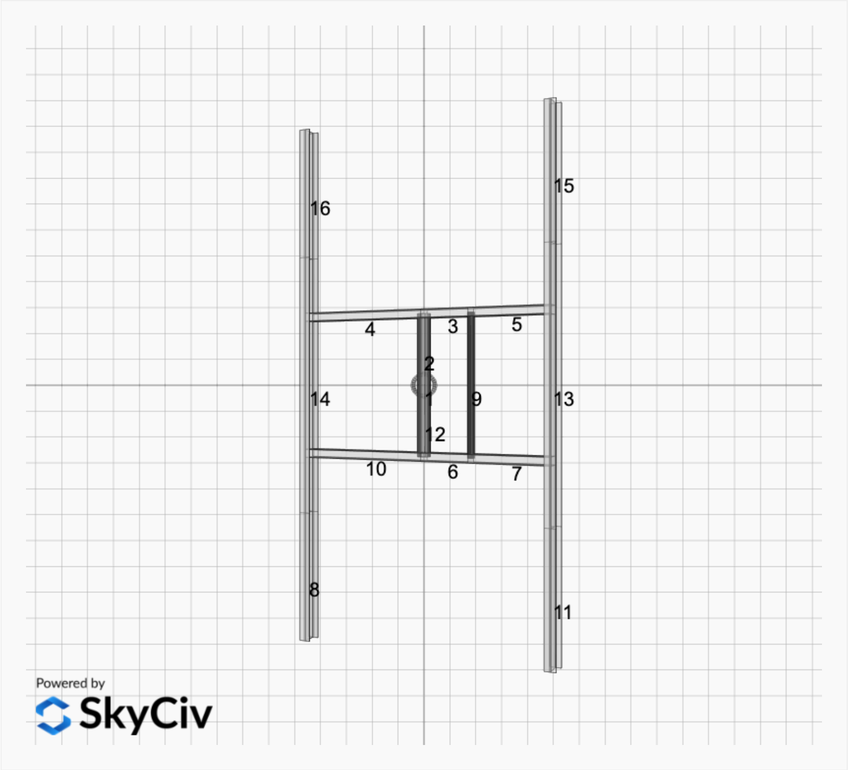

```
{"product_type":"Beam","project_id":"Stepman_JB_RevA","site_address":"16891 Lively Wood Ln, Rough and Ready, CA 95975, USA","module_width":41.2,"module_length":61.3,"number_rows":"4","number_columns":3,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":25,"ground_clearance":6,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"8_40","soil_type":"sand","foundation_type":"Round","foundation_size":36,"check_rails":true,"wind_speed_override":110,"snow_load_override":29,"direct_snow_load":false}
```

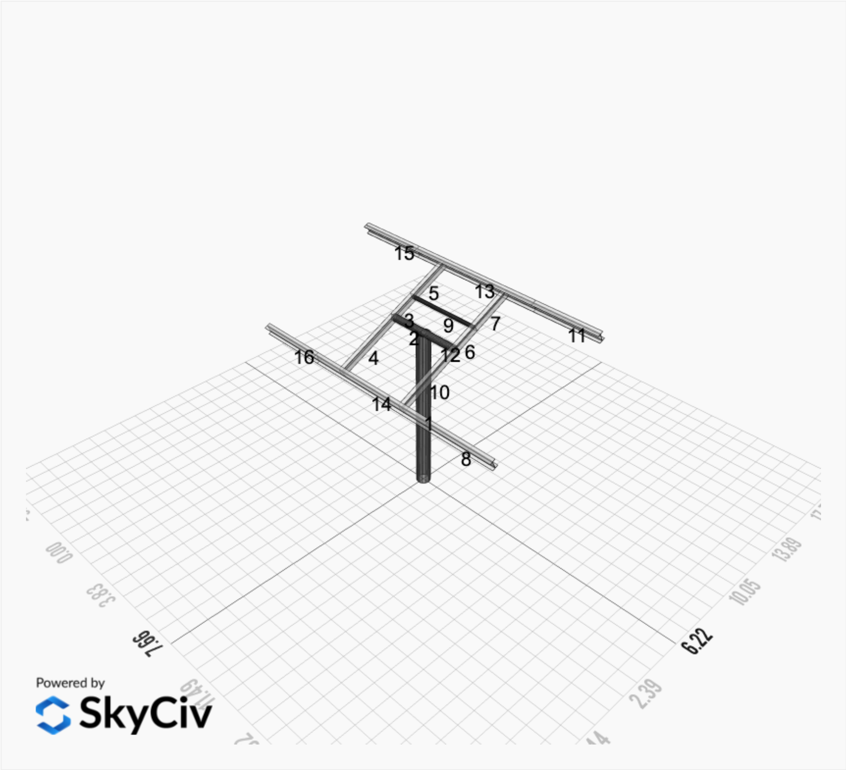

Design Notes:

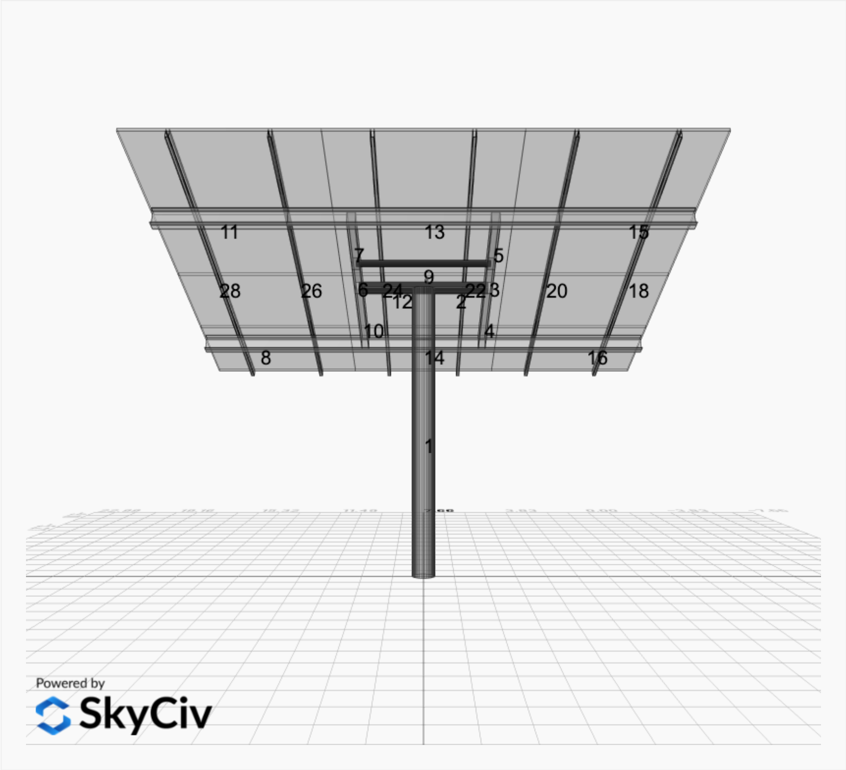
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



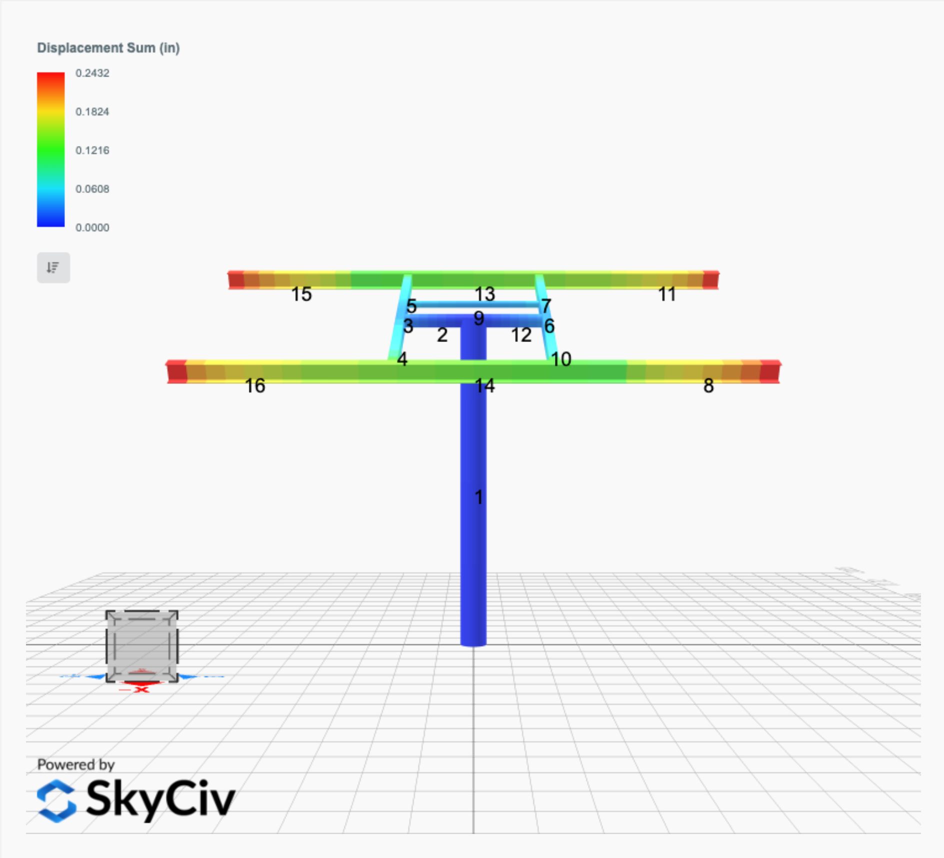

Powered by
SkyCiv

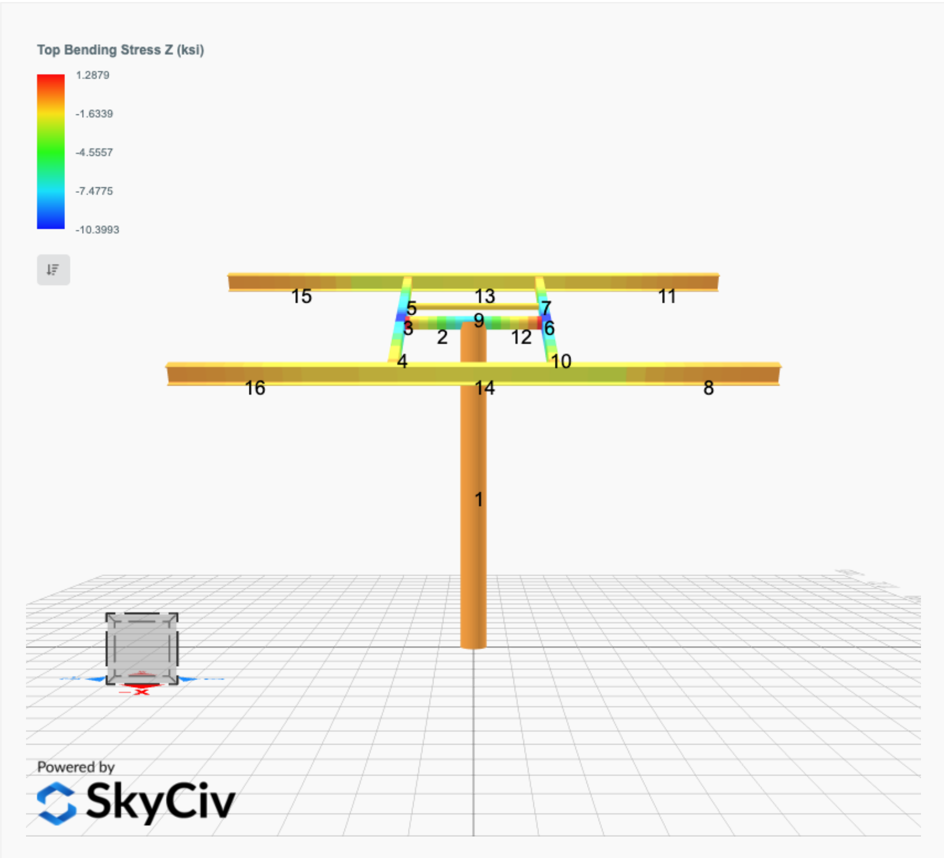

16
15
4
3
5
2
14
1
9
13
12
10
6
7
8
11
Powered by
SkyCiv

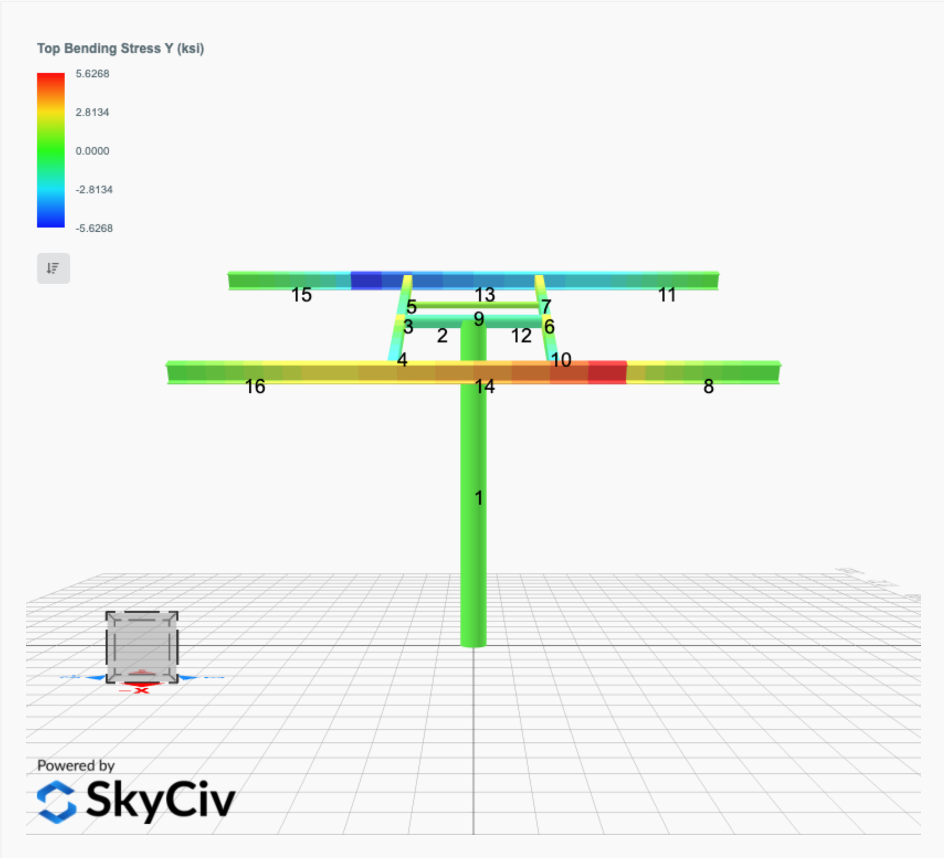

15
5
13
3
9
7
11
2
12
6
16
4
10
14
1
8
0.00
2.83
7.66
6.22
13.89
10.05
2.39
Powered by
SkyCiv

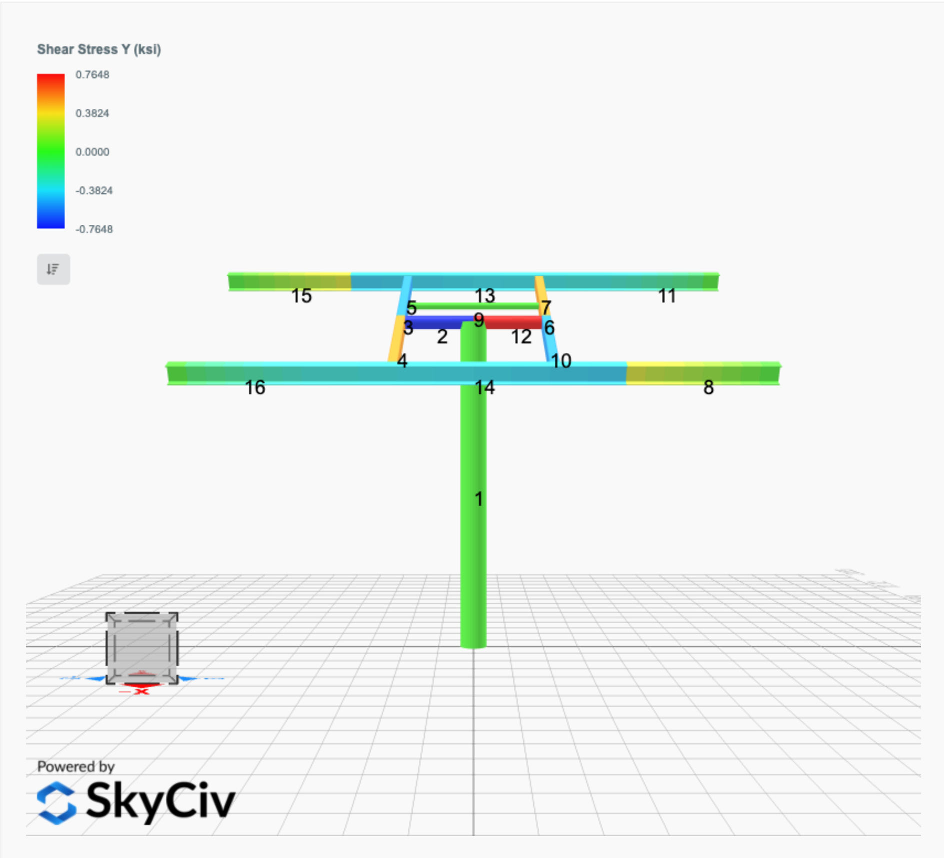

11
13
15
7
5
9
28
26
6
24
12
2
22
3
20
18
10
4
8
14
16
1
Powered by
SkyCiv

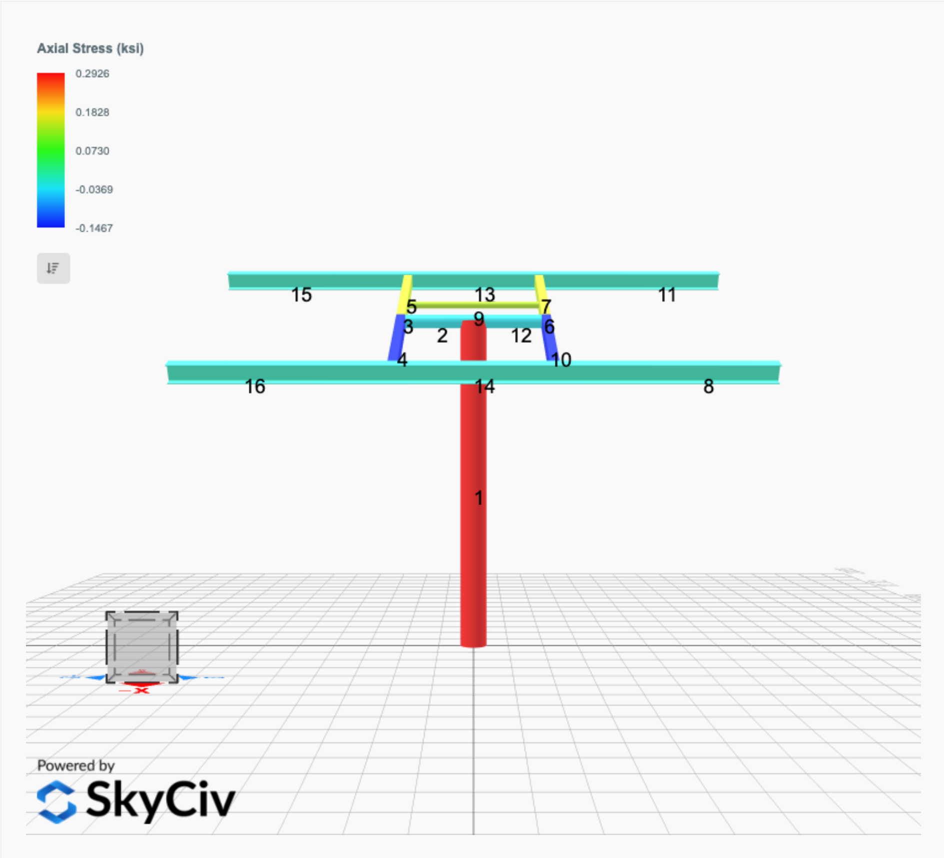
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
5.6268
2.8134
0.0000
-2.8134
-5.6268
15
13
11
5
9
7
3
2
12
6
4
10
16
14
8
1
Powered by
SkyCiv


Shear Stress Y (ksi)
0.7648
0.3824
0.0000
-0.3824
-0.7648
15
13
11
5
9
7
3
2
12
6
4
10
16
14
8
1
Powered by
SkyCiv


Axial Stress (ksi)
0.2926
0.1828
0.0730
-0.0369
-0.1467
15
13
11
5
7
3
9
6
2
12
4
10
16
14
8
1
Powered by
SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.7232	-0.0000	0.0000	-0.0000	0.0205
ULS: 2. D + L	0.0000	1.7232	-0.0000	0.0000	-0.0000	0.0205
ULS: 3. D + (S or Lr or R)	0.0000	4.1808	0.0000	0.0000	-0.0000	0.0289
ULS: 3. D + (S or Lr or R)	0.0000	1.7232	-0.0000	0.0000	-0.0000	0.0205
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.5664	0.0000	0.0000	-0.0000	0.0268
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.7232	-0.0000	0.0000	-0.0000	0.0205
ULS: 5b. D + 0.7E	0.0000	1.7232	-0.0000	0.0000	-0.0000	0.0205
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.5664	0.0000	0.0000	-0.0000	0.0268
ULS: 8. 0.6D + 0.7E	0.0000	1.0339	-0.0000	0.0000	-0.0000	0.0123
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.8070	5.5982	0.0000	0.0000	-0.0000	16.6541
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.8070	5.5982	0.0000	0.0000	-0.0000	16.6541
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5811	-1.6675	-0.0000	0.0000	-0.0000	-13.6843
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.3552	-1.1831	-0.0000	0.0000	-0.0000	-20.7450
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3552	6.4727	0.0000	0.0000	-0.0000	12.5020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3552	6.4727	0.0000	0.0000	-0.0000	12.5020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1858	1.0234	0.0000	0.0000	-0.0000	-10.2518
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0164	1.3867	-0.0000	0.0000	-0.0000	-15.5473
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3552	4.6294	0.0000	0.0000	-0.0000	12.4957
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.3552	4.6294	0.0000	0.0000	-0.0000	12.4957
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1858	-0.8198	-0.0000	0.0000	-0.0000	-10.2581
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0164	-0.4566	-0.0000	0.0000	-0.0000	-15.5536
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.8070	4.9089	0.0000	0.0000	-0.0000	16.6459
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.8070	4.9089	0.0000	0.0000	-0.0000	16.6459
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5811	-2.3568	-0.0000	0.0000	-0.0000	-13.6925
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.3552	-1.8724	-0.0000	0.0000	-0.0000	-20.7532

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value
Axial	9.7550
Shear X	-3.0116
Shear Z	0.0000
Moment X	-0.0000
Moment Z	34.8503

Worst Case Reactions ASD

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

Result	Value
Axial	6.4727
Shear X	-1.8070
Shear Z	-0.0000
Moment X	0.0000
Moment Z	20.7532

Project Details

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

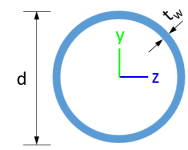
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

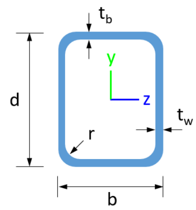
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

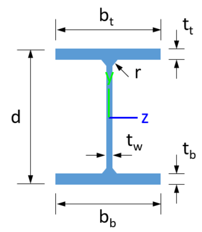
Section Dimensions



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					
9	8in Pipe Sch 40	8.63	0.32					



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	152.35	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	9	18.77	18.77	8.94	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.21,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,2.12,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.70,1.67,1.67,1.66,1.61	300	200	1
5	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.71,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
6	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.21,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.71,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	18	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,2.12,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.70,1.67,1.67,1.66,1.61	300	200	1
11	18	7.88	7.88	3.75	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	1.14	1.14	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
14	18	1.14	1.14	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
15	18	7.88	7.88	3.75	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	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
17	18	2.60	2.60	4.00	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.05,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10	300	200	1
18	18	1.14	1.14	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	300	200	1
19	18	2.60	2.60	4.00	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.17,1.09,1.09,1.09,1.00,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.14,1.09,1.09,1.09,1.04	300	200	1
20	18	1.14	1.14	1.75	1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30,1.30	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	377.97	245.94	83.29	83.29	113.39	113.39

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	54.44	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	54.44	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	118.62	23.36	6.45	30.09	45.74
14	120.60	118.62	23.36	6.45	30.09	45.74
15	120.60	54.44	23.36	6.45	30.09	45.74
16	120.60	54.44	23.36	6.45	30.09	45.74
17	120.60	110.58	23.36	6.45	30.09	45.74
18	120.60	118.62	23.36	6.45	30.09	45.74
19	120.60	110.58	22.29	6.45	30.09	45.74
20	120.60	118.62	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	0.040	0.418	0.000	0.027	0.000	0.421	#32	0.383	Not Required	Pass
2	0.002	0.522	0.191	0.110	0.035	0.714	#13	0.034	Not Required	Pass
3	0.008	0.844	0.079	0.085	0.008	0.888	#13	0.044	Not Required	Pass
4	0.007	0.819	0.176	0.083	0.022	0.884	#13	0.078	Not Required	Pass
5	0.008	0.523	0.182	0.085	0.026	0.546	#13	0.073	Not Required	Pass
6	0.008	0.844	0.079	0.085	0.008	0.888	#13	0.044	Not Required	Pass
7	0.008	0.523	0.182	0.085	0.026	0.546	#13	0.073	Not Required	Pass
8	0.000	0.096	0.085	0.040	0.006	0.170	#21	Not Required	Not Required	Pass
9	0.007	0.086	0.044	0.001	0.000	0.131	#13	0.198	Not Required	Pass
10	0.007	0.819	0.176	0.083	0.022	0.884	#13	0.078	Not Required	Pass
11	0.000	0.098	0.085	0.041	0.006	0.171	#21	Not Required	Not Required	Pass
12	0.002	0.522	0.191	0.110	0.035	0.714	#13	0.034	Not Required	Pass
13	0.000	0.212	0.182	0.060	0.009	0.368	#21	Not Required	Not Required	Pass
14	0.000	0.206	0.182	0.058	0.009	0.366	#21	Not Required	Not Required	Pass
15	0.000	0.098	0.085	0.041	0.006	0.171	#21	Not Required	Not Required	Pass
16	0.000	0.096	0.085	0.040	0.006	0.170	#21	Not Required	Not Required	Pass
17	0.004	0.222	0.052	0.022	0.003	0.250	#21	0.115	Not Required	Pass
18	0.000	0.212	0.182	0.060	0.009	0.368	#21	Not Required	Not Required	Pass
19	0.003	0.221	0.057	0.021	0.003	0.255	#21	0.172	Not Required	Pass
20	0.000	0.206	0.182	0.058	0.009	0.366	#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_{yy}	Moment of inertia about the Y axes

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

REFERENCES	CALCULATIONS	RESULTS

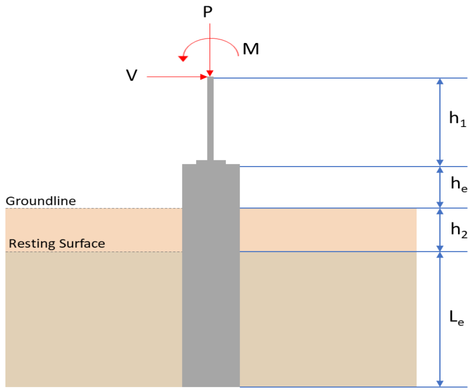
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 36 \text{ in}$ - Pile diameter
$L = 8.25 \text{ ft}$ - Total pile length
$h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile,
$h_2 = 0 \text{ ft}$ - Depth to resisting surface
$h_e = 0 \text{ ft}$ - 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)	6.473	9.755
V_x (kip)	-1.807	-3.012
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	20.753	34.850

Material Properties

$f'_{ck} = 3 \text{ ksi}$ - 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,

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

$$H_o = \frac{(-1.807 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.60233 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.753\ \text{kipft}) + ((-1.807\ \text{kip}) \times (0\ \text{ft}))}{(36\ \text{in})}$$

$$M_o = 6.9177\ \text{kipft/ft}$$

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:
$L_{e,x} = 7.5949\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(7.5949\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 7.595\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 8.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.595\ \text{ft})}{(8.25\ \text{ft})}$$

$$Ratio = 0.92061$$

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

End-bearing Capacity (ASD)

A - Pile cros-section area

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

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

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

q - End-bearing pressure

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

$$q = \frac{(6.473\ \text{kip})}{(7.0686\ \text{ft}^2)}$$

$$q = 0.91574\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.91574\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.45787$$

Status: **PASS**
Ratio: **0.460**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25\ \text{ft})}{(36\ \text{in})}$$

$$L/D = 2.75$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.60233\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.9177\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.9177\ \text{kipft/ft}) \times (8.25\ \text{ft})) + \left(3 \times (-0.60233\ \text{kip/ft}) \times (8.25\ \text{ft})^2\right)}{(6 \times (6.9177\ \text{kipft/ft})) + (4 \times (-0.60233\ \text{kip/ft}) \times (8.25\ \text{ft}))}$$

$$a = 5.7226\ \text{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (6.9177\ \text{kipft/ft})) + (3 \times (-0.60233\ \text{kip/ft}) \times (8.25\ \text{ft}))]^2}{(8.25\ \text{ft})^2 \times [(3 \times (6.9177\ \text{kipft/ft})) + (2 \times (-0.60233\ \text{kip/ft}) \times (8.25\ \text{ft}))]}$$

$$p = 0.26069\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (6.9177\ \text{kipft/ft})) + ((-0.60233\ \text{kip/ft}) \times (8.25\ \text{ft}))]}{(8.25\ \text{ft})^2}$$

$$s = 1.2277\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(5.7226\ \text{ft})}{2}$$

$$p_a = 0.4292\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.26069\ \text{kip/ft}^2)}{(0.4292\ \text{kip/ft}^2)}$$

$$Ratio = 0.6074$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.25\ \text{ft})$$

$$p_s = 1.2375\ \text{kip/ft}^2$$

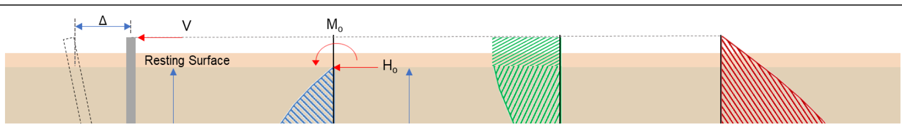
$Ratio$ - Lateral soil capacity

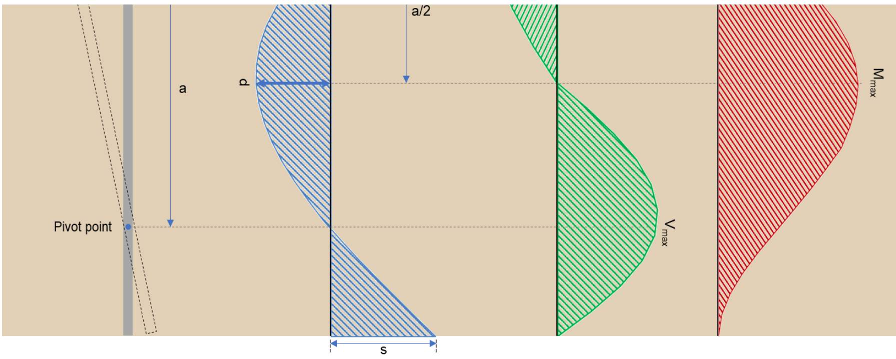
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(1.2277\ \text{kip/ft}^2)}{(1.2375\ \text{kip/ft}^2)}$$

$$Ratio = 0.99211$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.012 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -1.004 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.85 \text{ kipft}) + ((-3.012 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 11.617 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.617 \text{ kipft/ft})}{(-1.004 \text{ kip/ft})}$$

$$E = 11.57 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.617 \text{ kipft/ft}) \times (8.25 \text{ ft})) + \left(3 \times (-1.004 \text{ kip/ft}) \times (8.25 \text{ ft})^2\right)}{(6 \times (11.617 \text{ kipft/ft})) + (4 \times (-1.004 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7215 \text{ ft}$$

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

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

$$V_{max} = ((-1.004 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.57 \text{ ft})}{(8.25 \text{ ft})} + 3\right) \times \left(\frac{(5.7215 \text{ ft})}{(8.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.57 \text{ ft})}{(8.25 \text{ ft})} + 2\right) \times \left(\frac{(5.7215 \text{ ft})}{(8.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 9.4608 \text{ kip}$$

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

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

$$M_{max} = ((-1.004 \text{ kip/ft}) \times (36 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(11.57 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.7215 \text{ ft})}{2 \times (8.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (11.57 \text{ ft})}{(8.25 \text{ ft})} + 3\right) \times \left(\frac{(5.7215 \text{ ft})}{2 \times (8.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.57 \text{ ft})}{(8.25 \text{ ft})} + 2\right) \times \left(\frac{(5.7215 \text{ ft})}{2 \times (8.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 36.776 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(9.755 \text{ kip})}{(0.65)\times(0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2))\right]$$

$$A_{st,required} = -44.873 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-44.873 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.99533$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

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

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

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

Summary:

	Main reinforcement: **6 - #5 (0.625 in)** Ties: **#3(0.375 in) - 10 in**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.85\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(1017.9\text{ in}^2\right) - \left(1.8408\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(1.8408\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 1492.5\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.755\text{ kip})}{(1492.5\text{ kip})}$$ $$Ratio = 0.0065361$$	Status: **PASS** Ratio: **0.010**
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:** $b_w = 36\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (36\text{ in})$$ $$d = 28.8\text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(28.8\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.71796$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000\text{ psi})} \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,max} = 203.86\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $P = 9.755\text{ kip} \rightarrow 9755\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + \frac{(9755\text{ lbf})}{6 \times (1017.9\text{ in}^2)}\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,a} = 83.199\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (36\text{ in}) \times (28.8\text{ in})$$ $$V_{c,b} = 237.06\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(203.86\text{ kip}), (83.199\text{ kip}), (237.06\text{ kip})\right]$$	

$$V_c = 83.199 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((83.199 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 78.89 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.4608 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.4608 \text{ kip})}{(78.89 \text{ kip})}$$

$$Ratio = 0.11992$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

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

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

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

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

	$$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3\ \mathrm{ksi}) \times \left(4580.4\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 632.67\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(67.947\ \mathrm{kipft}), (632.67\ \mathrm{kipft})]$$ $$\phi M_n = 67.947\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 36.776\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(36.776\ \mathrm{kipft})}{(67.947\ \mathrm{kipft})}$$ $$Ratio = 0.54125$$		Status: **PASS** Ratio: **0.540**