Your Project Calculations

Project Name: South Pole Solar 20 mod 7 ft clearance

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=South%20Pole%20Solar%2020%20mod%207%20ft%20clearance&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=VUorCsce2PY0MQMZEbC3jW8nbewgDHh0hsjOVrr6vMlklEj3PaUBiYiLfoFjoaju

Array Specification

Product:	Beam
Unique ID:	1P-0-6TOP-XD-84-L-5Hx4W-ACD1
Duty Classification:	XD
Module Width:	44.64 in
Module Length:	67.87in
Number of Rows:	5
Number of Columns:	4
Total Number of Modules:	20
Desired Tilt Angle:	15
Front Edge Clearance:	7
Total Array Height at Tilt:	11.84 ft
Total Frame Length:	21.50 ft
Frame Weight:	968 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	22.96 ft
Rail Length:	225.70 in
Rail Spacing:	2.83 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	9.43 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.50 ft
Foundation Volume:	3.259 y^3
Foundation Result:	PASSED
Mount Twist:	0.000002 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	2245 Sean Ln, West Harrison, IN 47060, USA
Wind Speed:	100 mph
Snow Load:	20 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012096 ksf

Design Disclaimer

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

AutoDesigner Input


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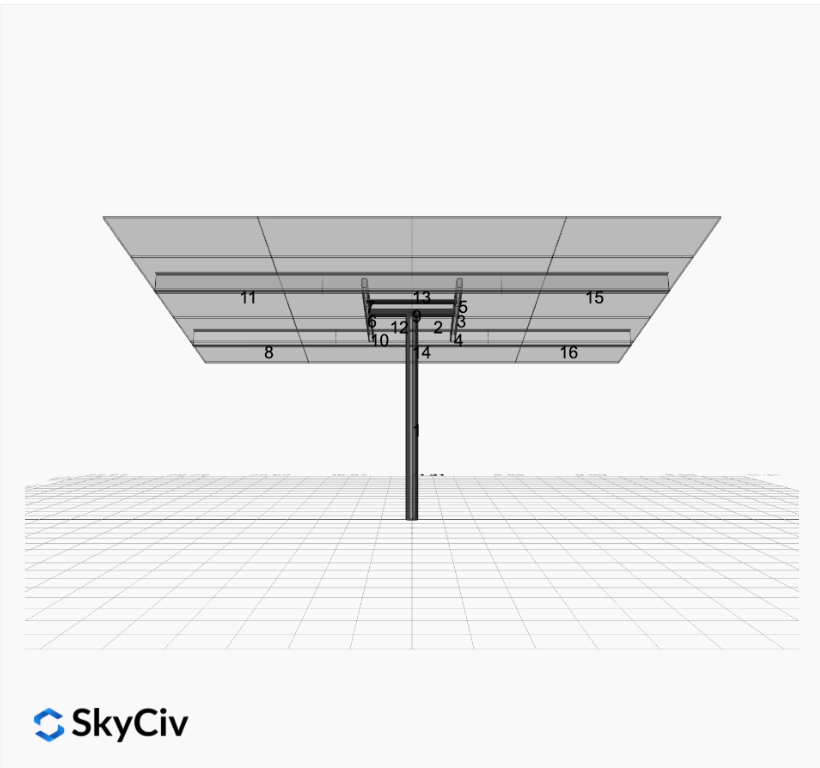
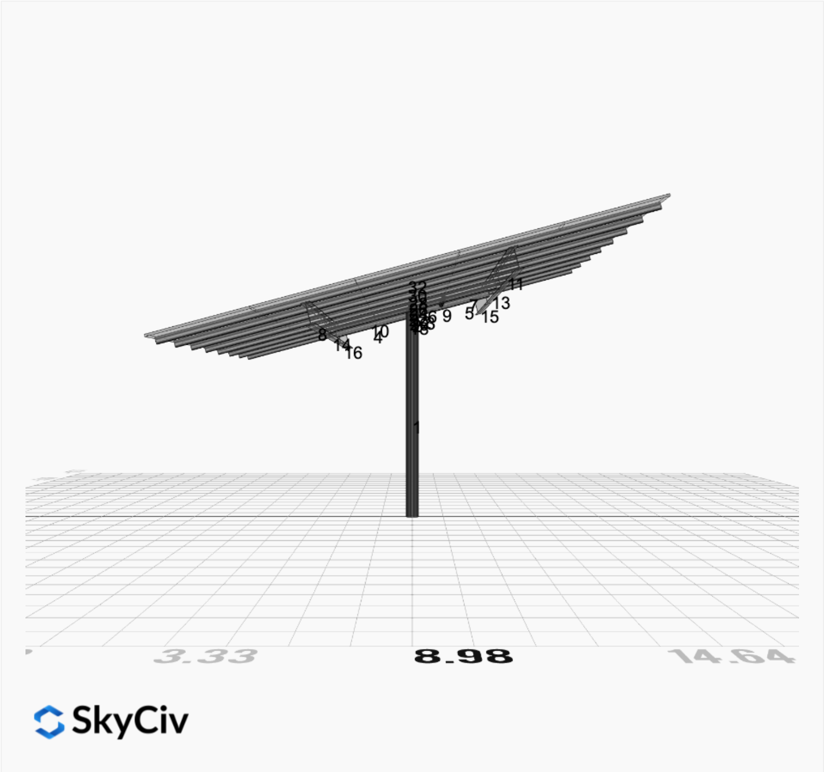
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles

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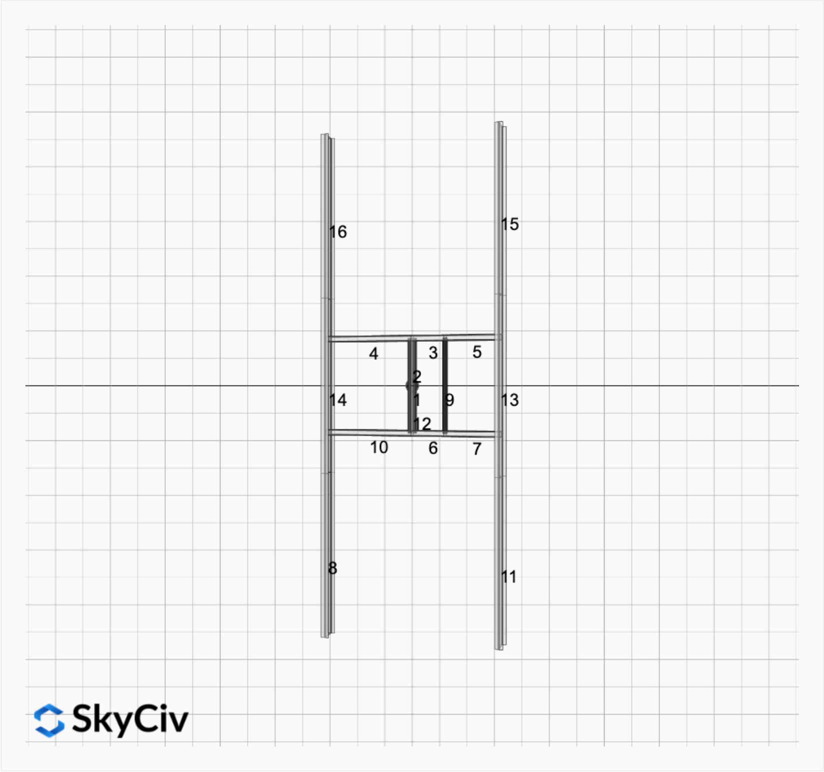
14.64
SkyCiv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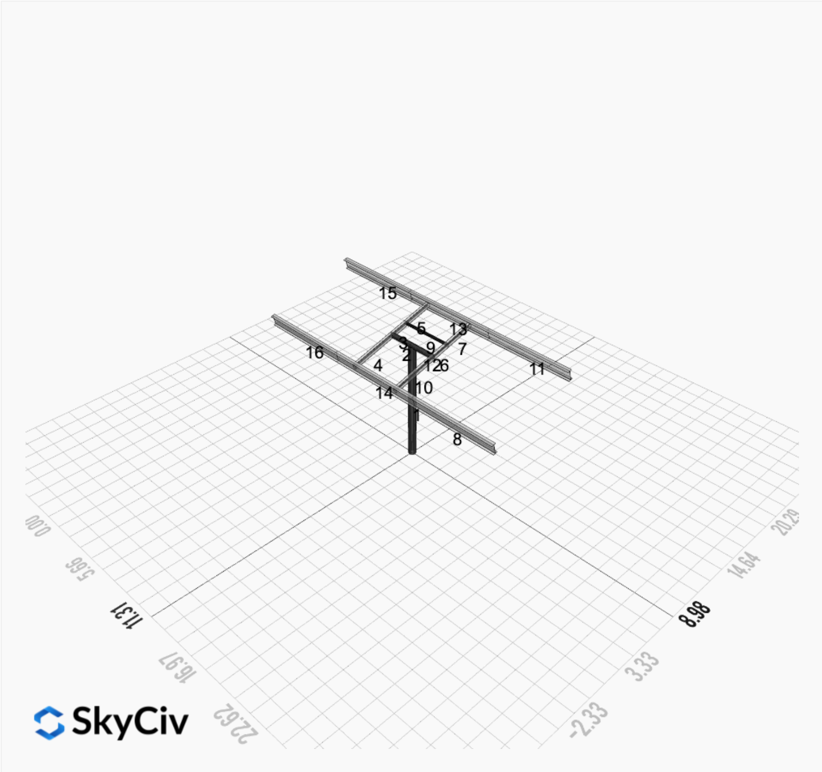
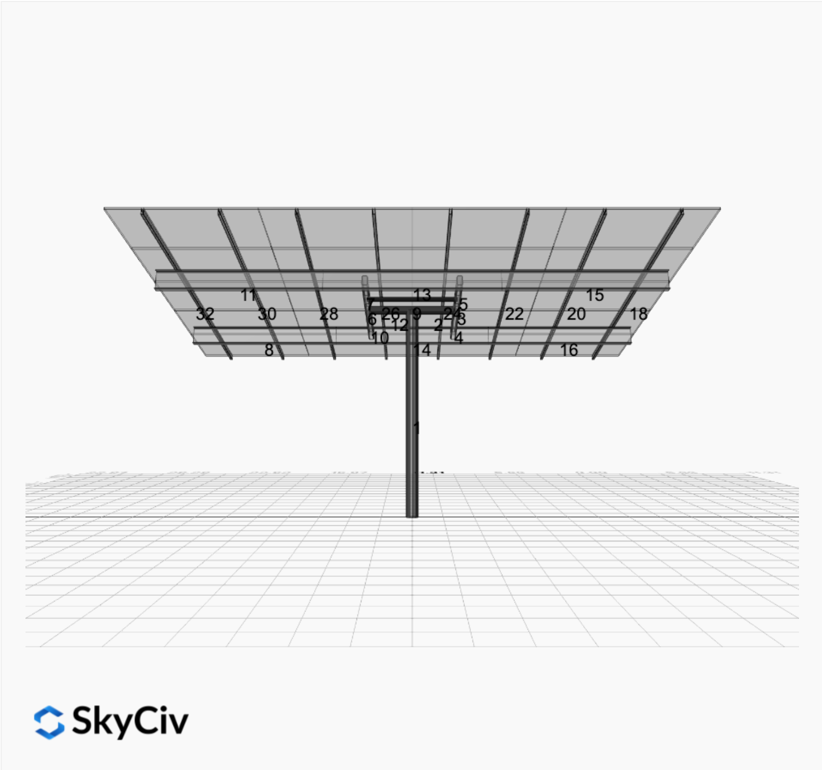
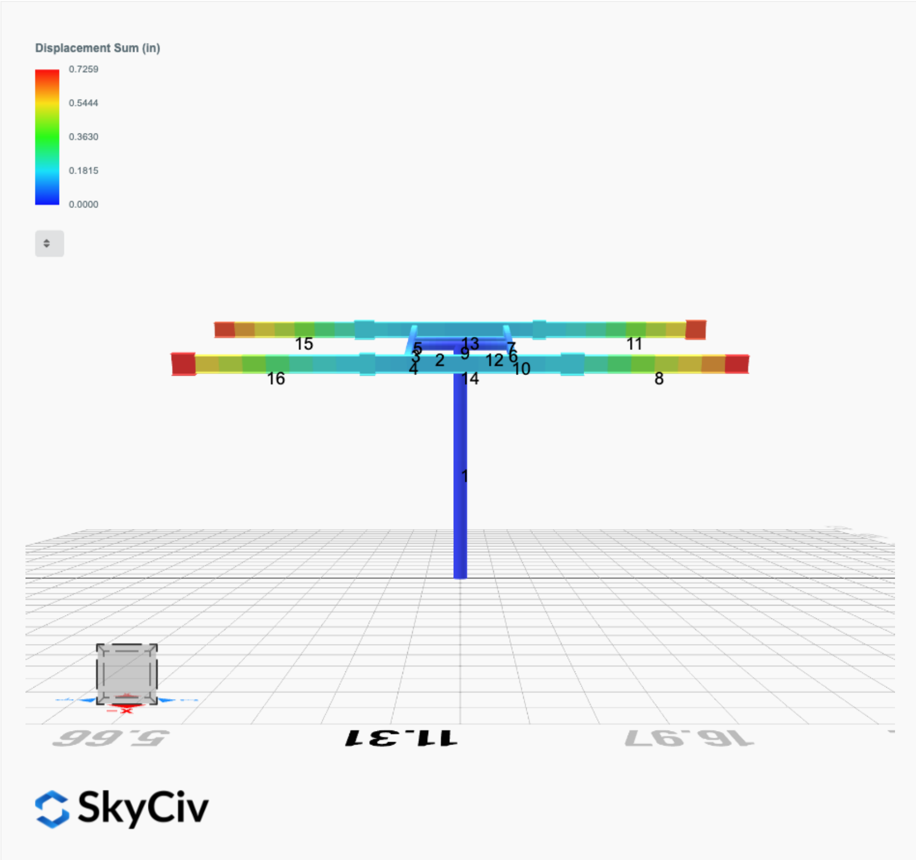
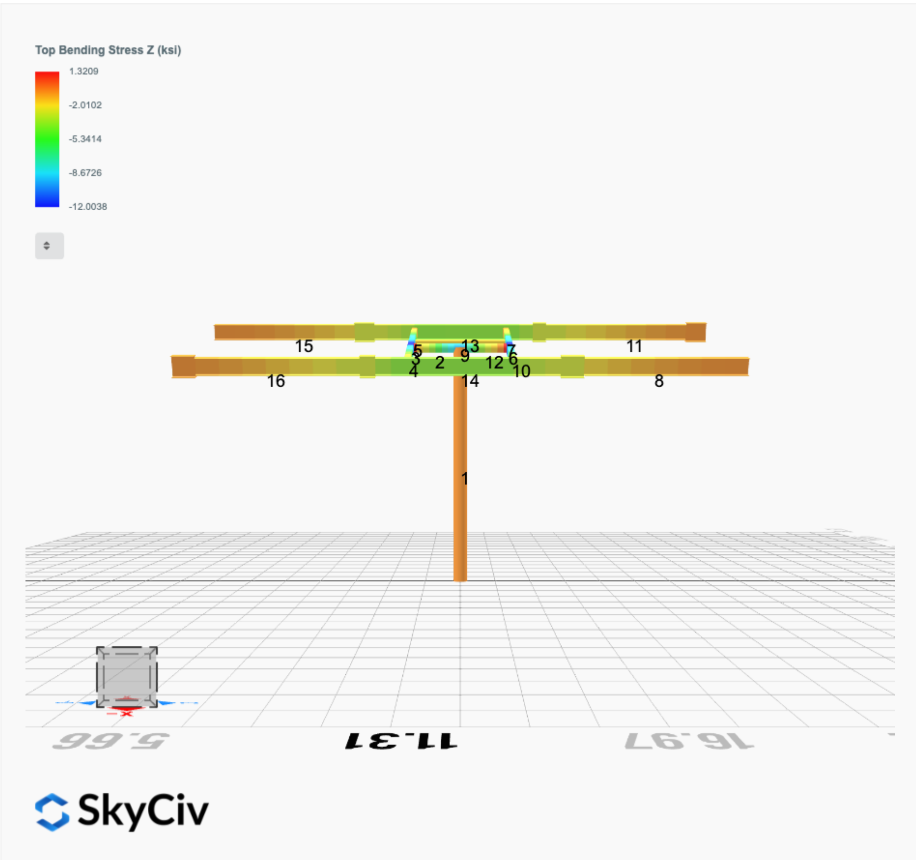
15
4
3
5
2
14
1
9
13
12
10
6
7
8
11
SkyCiv

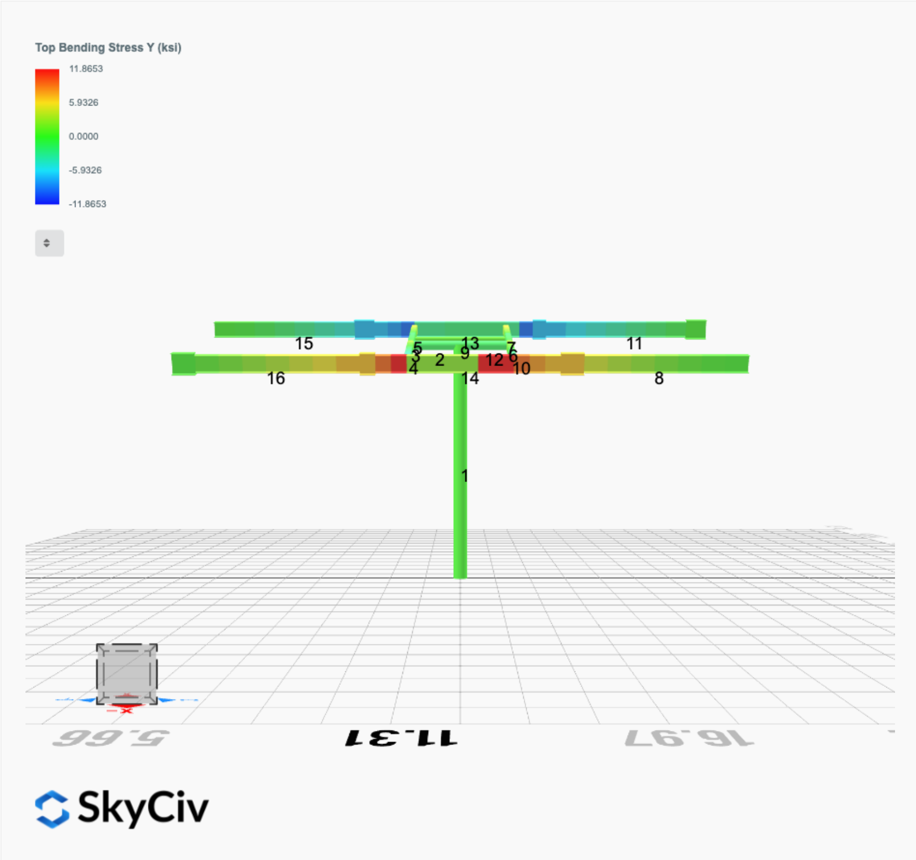




FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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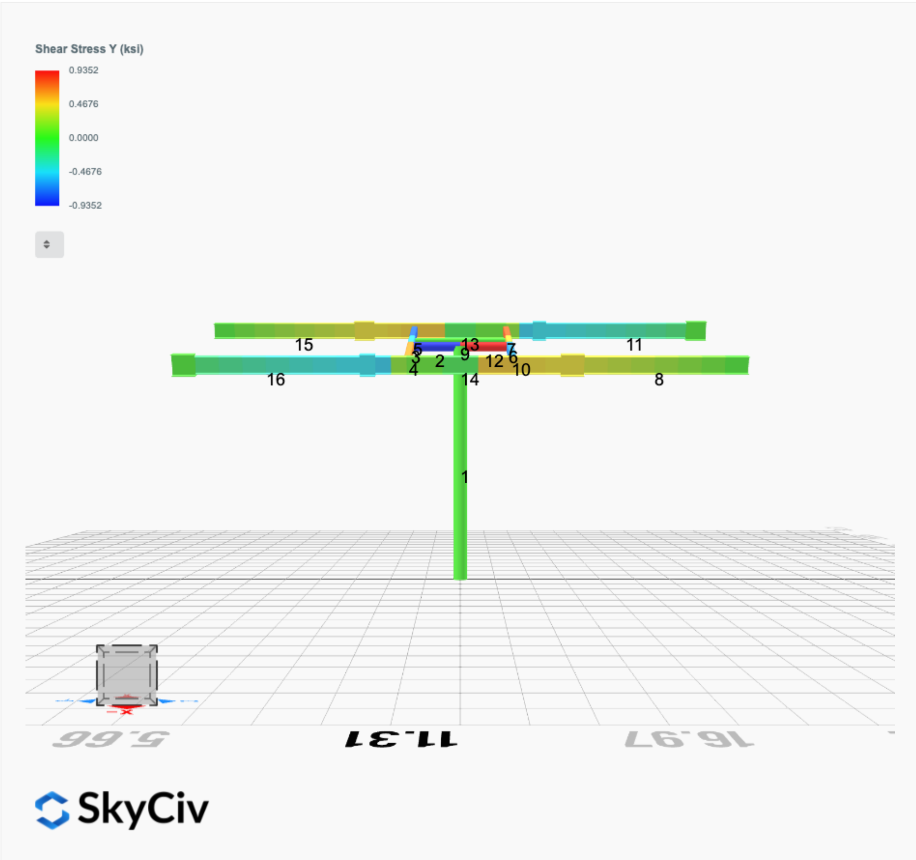
5.9326
0.0000
-5.9326
-11.8653
15
13
7
11
5
3
2
9
12
6
4
14
10
16
8
1
SkyCiv


Shear Stress Y (ksi)
0.9352
0.4676
0.0000
-0.4676
-0.9352
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13
7
11
5
3
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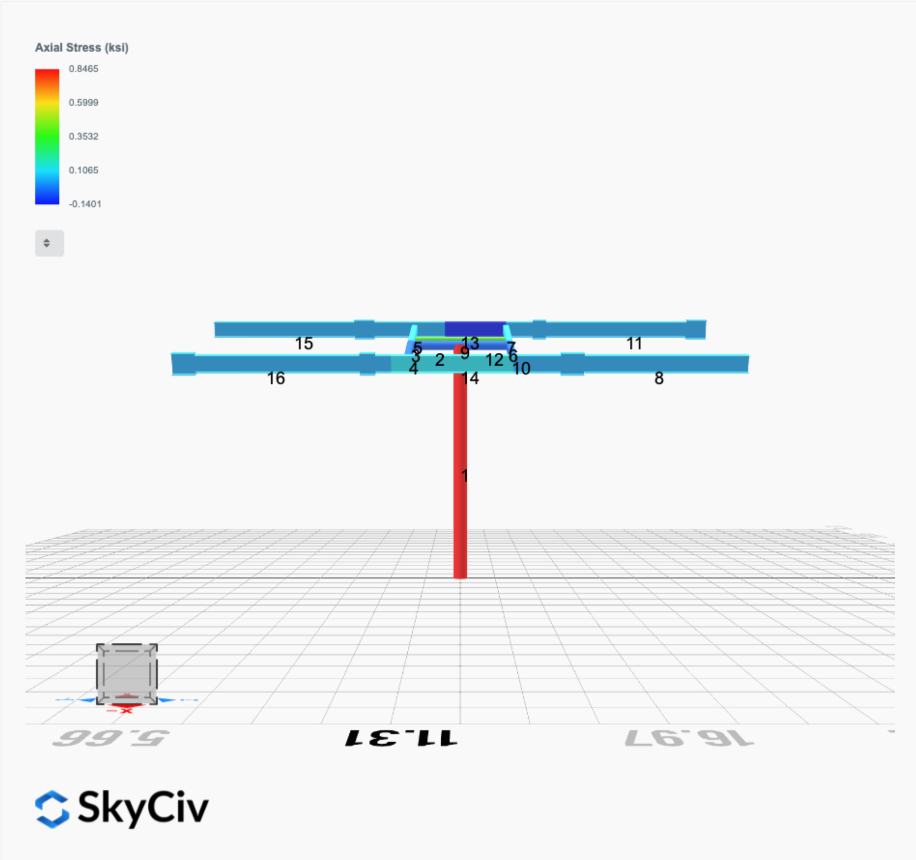
10
16
8
1
SkyCiv


Axial Stress (ksi)
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0.5999
0.3532
0.1065
-0.1401
15
11
5
13
7
3
2
9
12
6
4
14
10
16
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	2.9349	-0.0000	0.0000	-0.0000	0.0335
ULS: 2. D + L	0.0000	2.9349	-0.0000	0.0000	-0.0000	0.0335
ULS: 3. D + (S or Lr or R)	0.0000	7.6596	-0.0000	0.0000	-0.0000	0.0453
ULS: 3. D + (S or Lr or R)	0.0000	2.9349	-0.0000	0.0000	-0.0000	0.0335
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.4784	-0.0000	0.0000	-0.0000	0.0424
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.9349	-0.0000	0.0000	-0.0000	0.0335
ULS: 5b. D + 0.7E	0.0000	2.9349	-0.0000	0.0000	-0.0000	0.0335
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.4784	-0.0000	0.0000	-0.0000	0.0424
ULS: 8. 0.6D + 0.7E	0.0000	1.7609	-0.0000	0.0000	-0.0000	0.0201
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0005	6.6689	-0.0000	0.0000	-0.0000	11.3683
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0005	6.6689	-0.0000	0.0000	-0.0000	11.3683
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7590	0.1022	-0.0000	0.0000	-0.0000	-5.0275
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6555	0.4885	0.0000	0.0000	-0.0000	-15.2233
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7504	9.2789	-0.0000	0.0000	-0.0000	8.5435
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7504	9.2789	-0.0000	0.0000	-0.0000	8.5435
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5693	4.3539	-0.0000	0.0000	-0.0000	-3.7534
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4916	4.6436	-0.0000	0.0000	-0.0000	-11.4002
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.7504	5.7354	-0.0000	0.0000	-0.0000	8.5346
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.7504	5.7354	-0.0000	0.0000	-0.0000	8.5346
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5693	0.8104	-0.0000	0.0000	-0.0000	-3.7623
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4916	1.1001	0.0000	0.0000	-0.0000	-11.4091
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0005	5.4949	-0.0000	0.0000	-0.0000	11.3549
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0005	5.4949	-0.0000	0.0000	-0.0000	11.3549
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7590	-1.0717	-0.0000	0.0000	-0.0000	-5.0409
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6555	-0.6855	0.0000	0.0000	-0.0000	-15.2367

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	14.1930
Shear X	-1.6675
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	26.4217

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	9.2789
Shear X	-1.0005
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	15.2367

Project Details

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

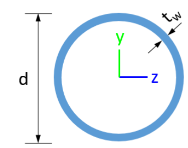
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

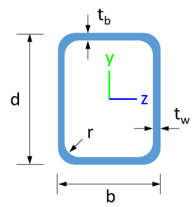
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

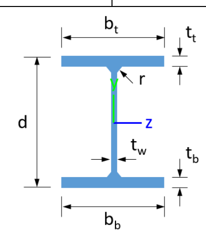
Section Dimensions



ID	Name	d (in)	t_w (in)
3	2in Pipe Sch 120	2.38	0.25
6	4in Pipe Sch 120	4.50	0.44
7	6in Pipe Sch 40	6.63	0.28



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38

20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	7	19.81	19.81	9.43	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.20,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.16,1.18,1.18,1.17,1.18	300	200	1
4	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.58,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,2.42,1.68,1.67,1.67,1.66,1.68	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.64,1.67,1.67,1.65,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.20,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.16,1.18,1.18,1.17,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.69,1.66,1.67,1.67,1.64,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.70,1.64,1.67,1.67,1.65,1.66	300	200	1
8	20	14.70	14.70	7.00	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	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.58,1.68,1.67,1.67,1.65,1.68,1.67,1.67,1.67,1.67,1.67,1.67,2.42,1.68,1.67,1.67,1.66,1.68	300	200	1
11	20	14.70	14.70	7.00	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	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.08,1.02,1.02,1.02,1.04,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.10,1.02,1.02,1.02,1.03	300	200	1
14	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.11,1.03,1.02,1.02,1.02,1.08,1.02,1.02,1.02,1.02,1.02,1.02,1.00,1.03,1.02,1.02,1.02,1.12	300	200	1
15	20	14.70	14.70	7.00	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	20	14.70	14.70	7.00	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	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	110.67	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	15.83	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	15.83	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	31.17	6.46	56.26	44.91

14	159.30	116.35	30.56	6.46	56.26	44.91
15	159.30	15.83	46.90	6.46	56.26	44.91
16	159.30	15.83	46.90	6.46	56.26	44.91

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	0.128	0.625	0.000	0.022	0.000	0.633	#16	0.529	Not Required	Pass
2	0.004	0.453	0.073	0.093	0.011	0.517	#21	0.036	Not Required	Pass
3	0.005	0.646	0.037	0.065	0.009	0.659	#21	0.046	Not Required	Pass
4	0.005	0.612	0.138	0.061	0.028	0.691	#21	0.082	Not Required	Pass
5	0.005	0.401	0.143	0.064	0.038	0.442	#21	0.076	Not Required	Pass
6	0.005	0.646	0.037	0.065	0.009	0.659	#21	0.046	Not Required	Pass
7	0.005	0.401	0.143	0.064	0.038	0.442	#21	0.076	Not Required	Pass
8	0.000	0.159	0.241	0.038	0.010	0.400	#21	Not Required	Not Required	Pass
9	0.021	0.074	0.028	0.001	0.000	0.112	#21	0.206	Not Required	Pass
10	0.005	0.612	0.138	0.061	0.028	0.691	#21	0.082	Not Required	Pass
11	0.000	0.167	0.241	0.040	0.010	0.408	#21	Not Required	Not Required	Pass
12	0.004	0.453	0.073	0.093	0.011	0.517	#21	0.036	Not Required	Pass
13	0.007	0.406	0.376	0.050	0.012	0.768	#21	0.204	Not Required	Pass
14	0.007	0.392	0.376	0.047	0.012	0.749	#21	0.204	Not Required	Pass
15	0.000	0.167	0.241	0.040	0.010	0.408	#21	Not Required	Not Required	Pass
16	0.000	0.159	0.241	0.038	0.010	0.400	#21	Not Required	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

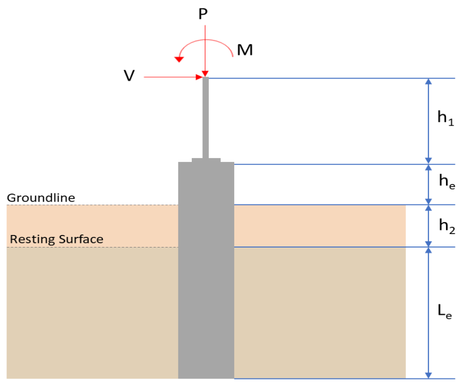
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 5.5\ \mathrm{ft}$ - 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) $(\mathrm{psf/ft})$
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.279	14.193
V_x (kip)	-1.001	-1.668
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	15.237	26.422

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,

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

$$H_o = \frac{(-1.001\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.15939\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.237\ \mathrm{kipft}) + ((-1.001\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$1.57 \times (48\ \mathrm{in})$$

$$M_o = 2.4263\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 5.2412\ \mathrm{ft}$ - 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,

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

$$L_{e,req} = MAX[(5.2412\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 5.241\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 5.5\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.241\ \mathrm{ft})}{(5.5\ \mathrm{ft})}$$

$$Ratio = 0.95291$$

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

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

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

$$q = \frac{(9.279\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.57994\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.57994\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.28997$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.15939\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.4263\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.4263\ \mathrm{kipft/ft}) \times (5.5\ \mathrm{ft})) + \left(3 \times (-0.15939\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft})^2\right)}{(6 \times (2.4263\ \mathrm{kipft/ft})) + (4 \times (-0.15939\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft}))}$$

$$a = 3.7556\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.4263\ \mathrm{kipft/ft})) + (3 \times (-0.15939\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft}))]^2}{(5.5\ \mathrm{ft})^2 \times [(3 \times (2.4263\ \mathrm{kipft/ft})) + (2 \times (-0.15939\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft}))]}$$

$$p = 0.22461\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.4263\ \mathrm{kipft/ft})) + ((-0.15939\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft}))]}{(5.5\ \mathrm{ft})^2}$$

$$s = 0.7886\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.7556\ \mathrm{ft})}{2}$$

$$p_a = 0.28167\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22461\ \mathrm{kip/ft^2}\right)}{\left(0.28167\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.79741$$

Status: **PASS**
Ratio: **0.800**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.5\ \mathrm{ft})$$

$$p_s = 0.825\ \mathrm{kip/ft^2}$$

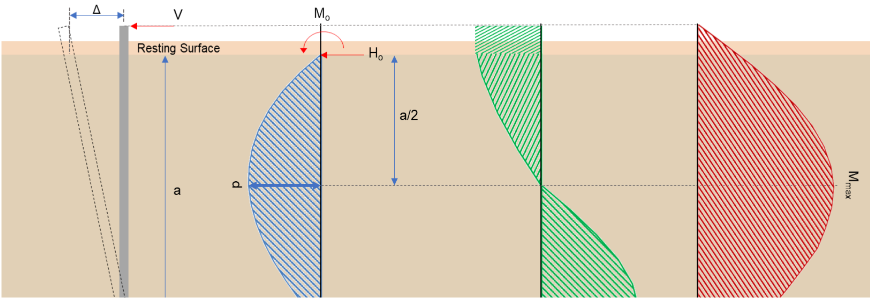
$Ratio$ - Lateral soil capacity

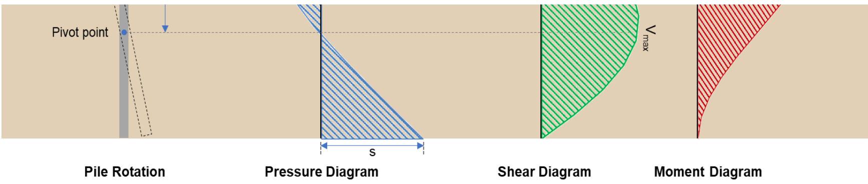
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.7886\ \mathrm{kip/ft^2}\right)}{\left(0.825\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.95588$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.668\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.26561\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.422\ \mathrm{kipft}) + ((-1.668\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.2073\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.2073\ \mathrm{kipft/ft})}{(-0.26561\ \mathrm{kip/ft})}$$

$$E = 15.841\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2073\ \mathrm{kipft/ft}) \times (5.5\ \mathrm{ft})) + \left(3 \times (-0.26561\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft})^2\right)}{(6 \times (4.2073\ \mathrm{kipft/ft})) + (4 \times (-0.26561\ \mathrm{kip/ft}) \times (5.5\ \mathrm{ft}))}$$

$$a = 3.7528\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.26561\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.841\ \mathrm{ft})}{(5.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(3.7528\ \mathrm{ft})}{(5.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.841\ \mathrm{ft})}{(5.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(3.7528\ \mathrm{ft})}{(5.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.1199\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.26561\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (5.5\ \mathrm{ft})) \times \left[\left(\frac{(15.841\ \mathrm{ft})}{(5.5\ \mathrm{ft})} + \frac{(3.7528\ \mathrm{ft})}{2 \times (5.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (15.841\ \mathrm{ft})}{(5.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(3.7528\ \mathrm{ft})}{2 \times (5.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (15.841\ \mathrm{ft})}{(5.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(3.7528\ \mathrm{ft})}{2 \times (5.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 16.296\ \mathrm{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(14.193\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.125\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.125\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(14.193 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0053054$$

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.193 \text{ kip} \rightarrow 14193 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14193 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.38 \text{ kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.38 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.38 \text{ kip}$$

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

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

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

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

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

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

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

$$\phi V_n = (0.65) \times ((120.38 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.33 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.1199 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.1199 \text{ kip})}{(111.33 \text{ kip})}$$

$$Ratio = 0.054972$$

Status: **PASS**
Ratio: **0.050**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

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

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

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

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

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

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

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

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:

$M_{max} = 16.296 \text{ kipft}$ - Maximum moment in the x-direction,

	$Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(16.296 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.065288$$	Status: **PASS** Ratio: **0.070**