Your Project Calculations

Project Name: Lewis Godby TOP15 Struts 25D

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Lewis%20Godby%20TOP15%20Struts%2025D&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=tjZ5wITteKIR3dqZYqHfAXMgkPR6rjZh4IgZ6Ufg3W4pNXZT5tGiGmGhnB7PwtUs

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-XD-84-L-5Hx3W-3A55
Duty Classification:	XD
Module Width:	44.65 in
Module Length:	88.80in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Desired Tilt Angle:	25
Front Edge Clearance:	8
Total Array Height at Tilt:	15.91 ft
Total Frame Length:	21.50 ft
Frame Weight:	1694 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	22.45 ft
Rail Length:	225.75 in
Rail Spacing:	3.70 ft
Rail Check:	Not Checked

Support Specifications

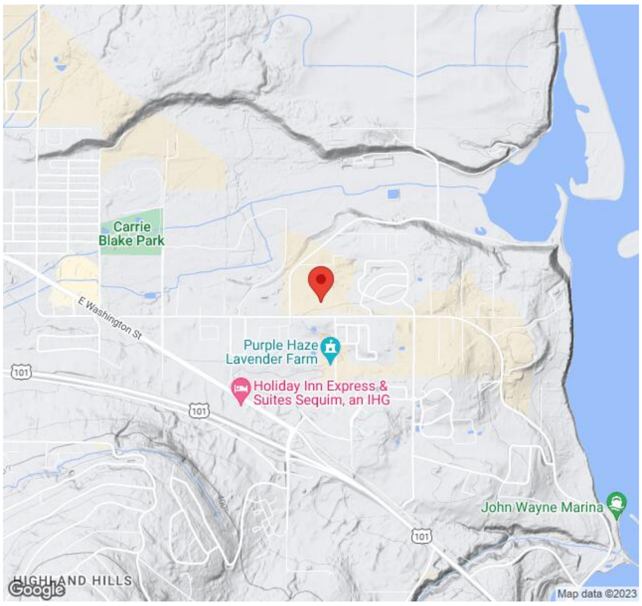
Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	11.98 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.75 ft
Foundation Volume:	4.593 y^3
Foundation Result:	PASSED
Mount Twist:	0.001630 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	855 W Sequim Bay Rd, Sequim, WA 98382, USA
Wind Speed:	110 mph
Snow Load:	25 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012371 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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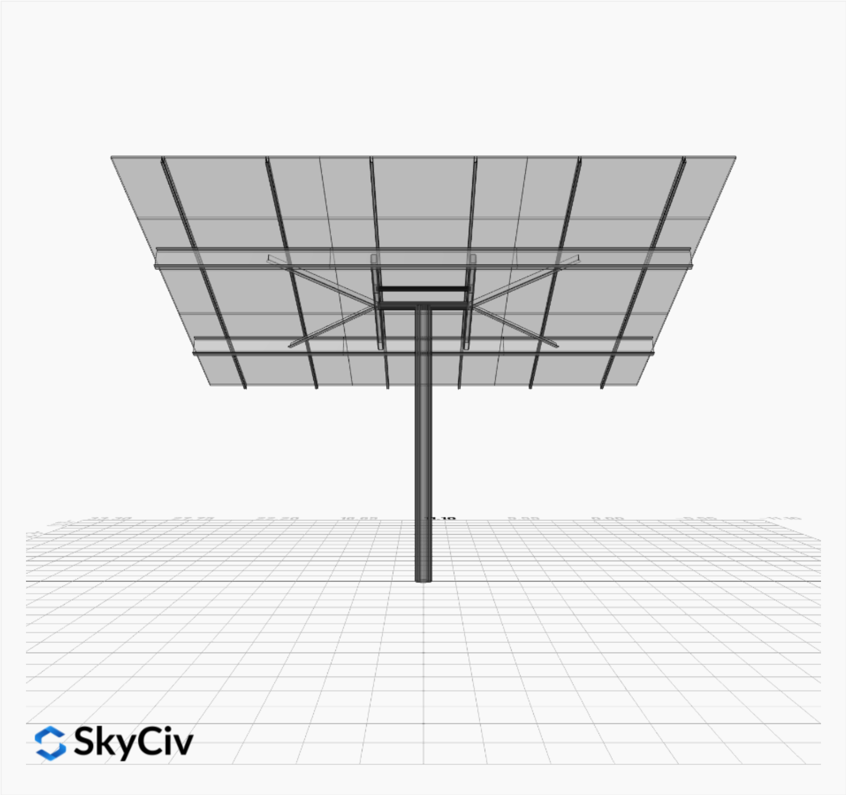
Design Notes:

- 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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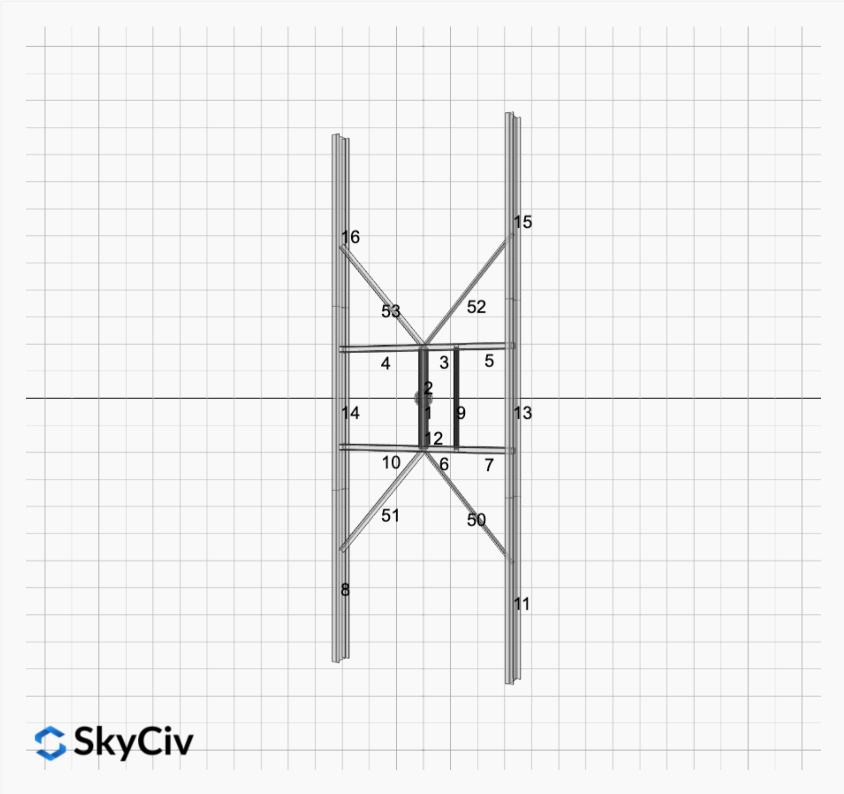
16
53
1
SkyCiv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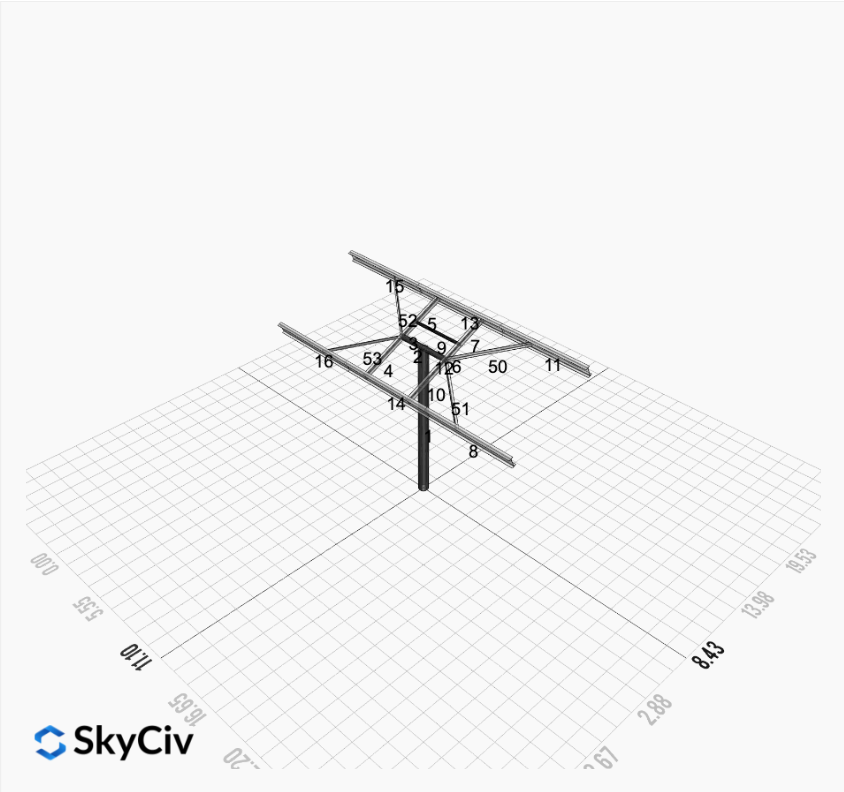
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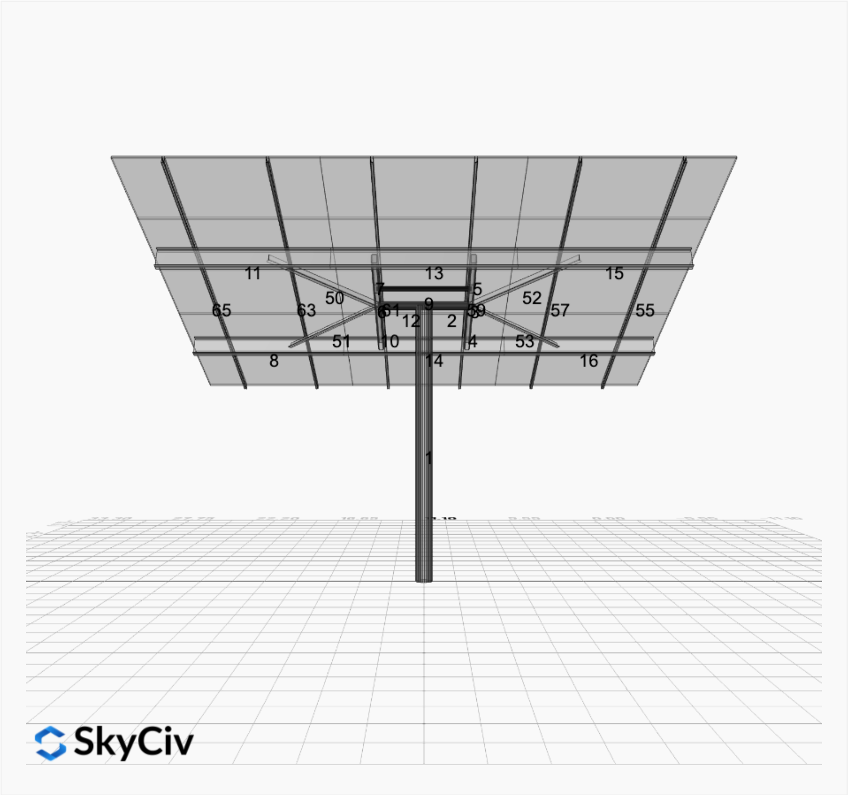
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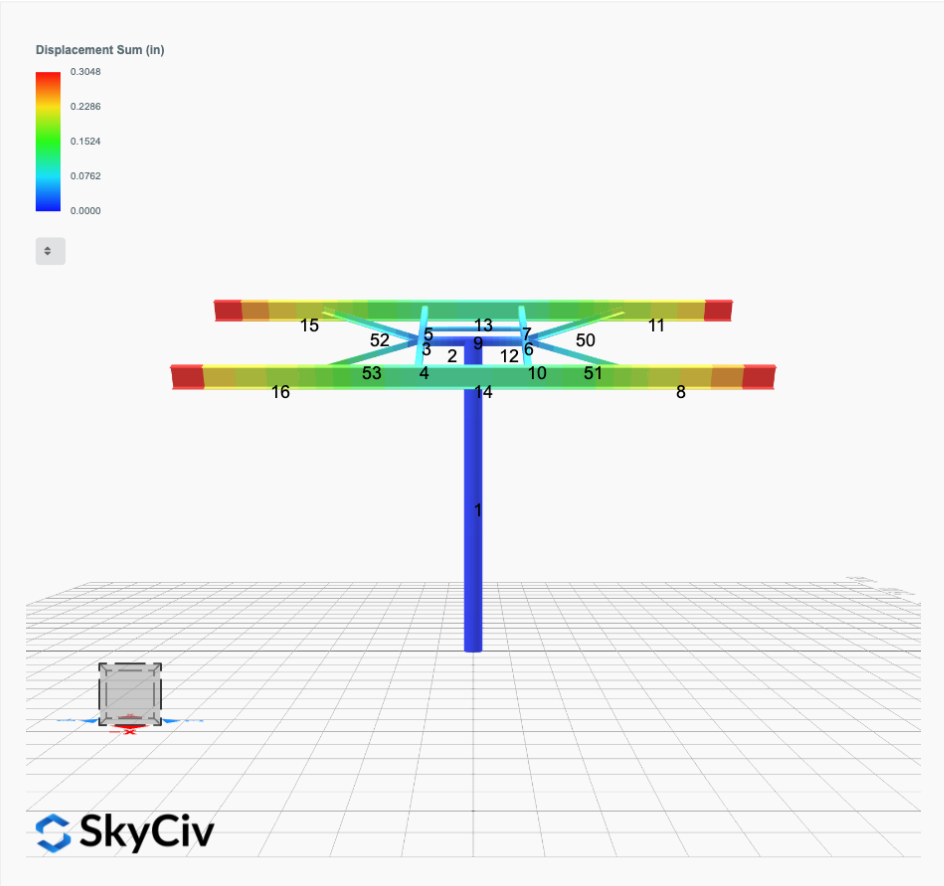
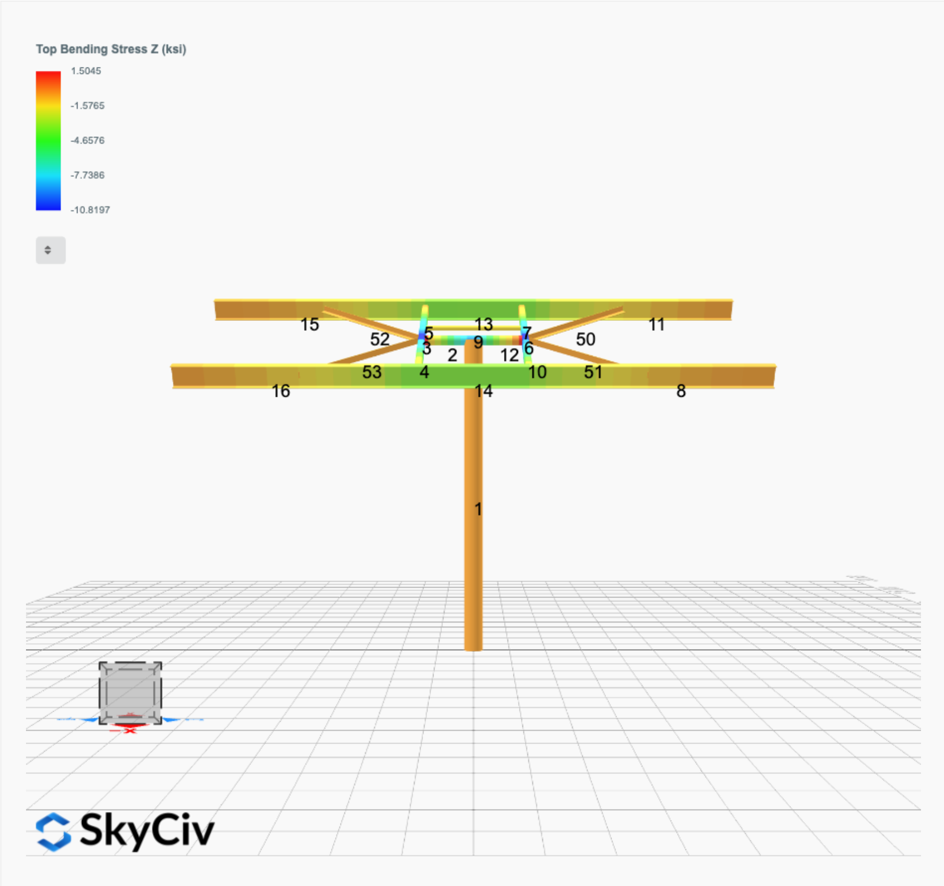
8
14
16
1
SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
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-1.5765
-4.6576
-7.7386
-10.8197
15
13
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7
50
3
9
6
2
12
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4
10
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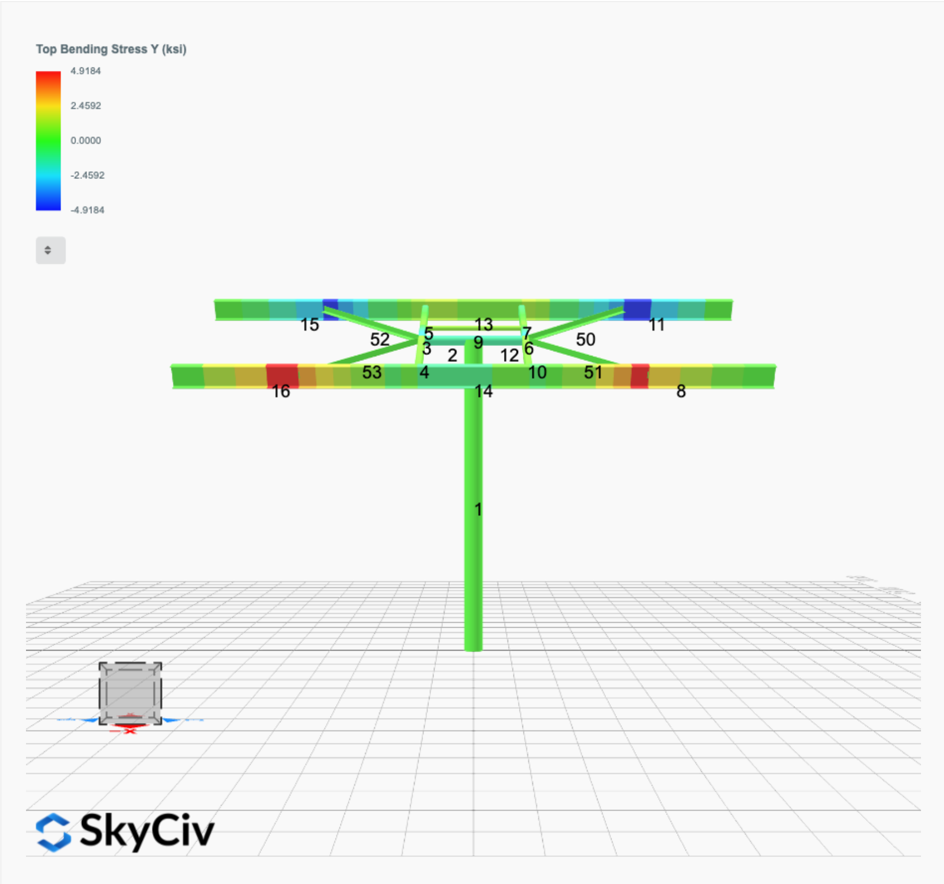
1
SkyCiv


Top Bending Stress Y (ksi)
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0.0000
-2.4592
-4.9184
15
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2
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10
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16
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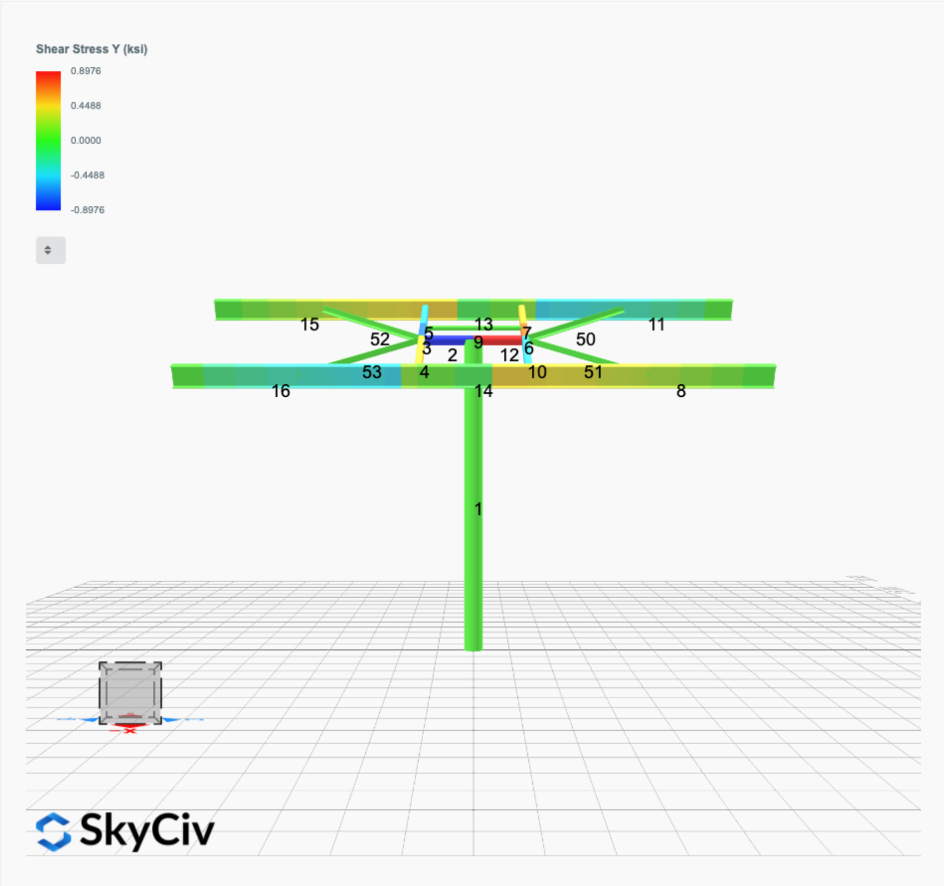
SkyCiv


Shear Stress Y (ksi)
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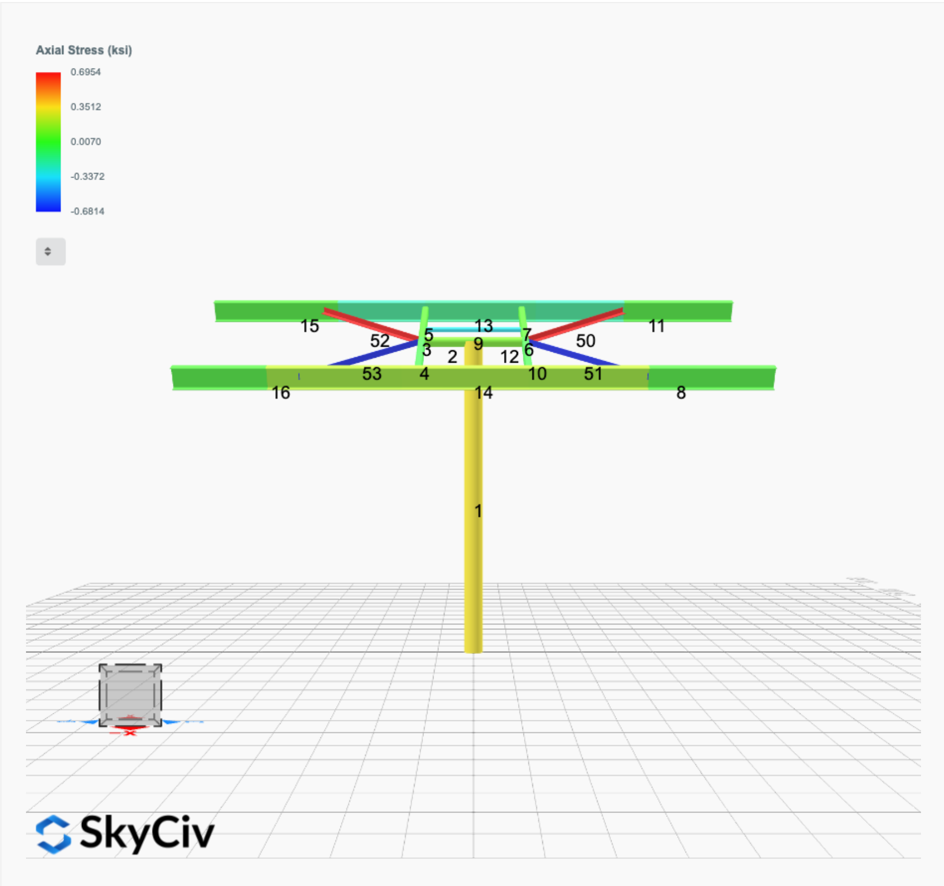
SkyCiv


Axial Stress (ksi)
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11
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3
9
6
2
12
53
4
10
51
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	3.2488	0.0000	-0.0000	-0.0000	0.0339
ULS: 2. D + L	0.0000	3.2488	0.0000	-0.0000	-0.0000	0.0339
ULS: 3. D + (S or Lr or R)	0.0000	7.7836	0.0000	-0.0000	-0.0000	0.0589
ULS: 3. D + (S or Lr or R)	0.0000	3.2488	0.0000	-0.0000	-0.0000	0.0339
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	6.6499	0.0000	-0.0000	-0.0000	0.0526
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.2488	0.0000	-0.0000	-0.0000	0.0339
ULS: 5b. D + 0.7E	0.0000	3.2488	0.0000	-0.0000	-0.0000	0.0339
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	6.6499	0.0000	-0.0000	-0.0000	0.0526
ULS: 8. 0.6D + 0.7E	0.0000	1.9493	0.0000	-0.0000	-0.0000	0.0204
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7777	11.3500	-0.0000	-0.0000	-0.0000	47.3225
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.7777	11.3500	-0.0000	-0.0000	-0.0000	47.3225
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3055	-3.8398	0.0000	-0.0000	-0.0000	-37.9566
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.8332	-2.8271	0.0000	-0.0000	-0.0000	-51.0844
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8332	12.7258	-0.0000	-0.0000	-0.0000	35.5190
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8332	12.7258	-0.0000	-0.0000	-0.0000	35.5190
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4791	1.3335	0.0000	-0.0000	-0.0000	-28.4402
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.1249	2.0930	0.0000	-0.0000	-0.0000	-38.2861
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8332	9.3247	-0.0000	-0.0000	-0.0000	35.5003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8332	9.3247	-0.0000	-0.0000	-0.0000	35.5003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4791	-2.0676	0.0000	-0.0000	-0.0000	-28.4589
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	2.1249	-1.3081	0.0000	-0.0000	-0.0000	-38.3048
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7777	10.0505	-0.0000	-0.0000	-0.0000	47.3089
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.7777	10.0505	-0.0000	-0.0000	-0.0000	47.3089
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3055	-5.1393	0.0000	-0.0000	-0.0000	-37.9701
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.8332	-4.1266	0.0000	-0.0000	-0.0000	-51.0980

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	19.6680
Shear X	-6.2961
Shear Z	0.0000
Moment X	0.0009
Moment Y (Twist)	0.0016
Moment Z	86.5799

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	12.7258
Shear X	-3.7777
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	51.0980

Project Details

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

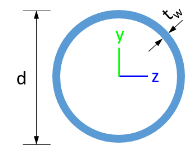
User Name: sales@mtsolar.us
Project Name: Lewis Godby TOP15 Struts 25D
Unit System: imperial

Design Input Information

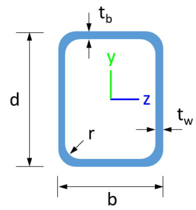
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

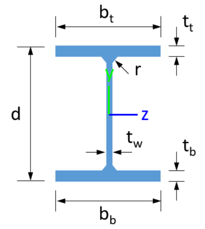
Section Dimensions



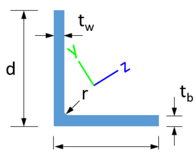
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
10	8in Pipe Sch 80	8.63	0.50					



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



b

ID	Name	d (in)	t_w (in)	b (in)	t_b (in)	r (in)		
34	L3x2x3/16	3.00	0.19	2.00	0.19	0.31		

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
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
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
34	L3x2x3/16	0.92	0.01	0.17	0.98	0.01	0.33	0.82

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	10	25.15	25.15	11.98	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.14,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.51,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.70,1.67,1.67,1.66,1.63	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.62,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.14,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.62,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	20	7.00	7.00	7.00	2.32,2.32,2.32,2.31,2.32,2.32,2.31,2.31,2.31,2.28,2.31,2.31,2.31,2.35,2.31,2.31,2.31,2.30,2.32,2.32,2.30,2.32,2.31,2.31,2.31,2.32	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.69,1.67,1.67,1.66,1.51,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.70,1.67,1.67,1.66,1.63	300	200	1
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12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.80,1.02,1.02,1.02,1.03,1.02,1.02,1.02,1.02	300	200	1
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50	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250
51	34	5.67	5.67	5.67	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14	300	200	250

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53	34	5.67	5.67	5.6 7	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.1 4,1.14,1.14,1.14,1.14,1.14,1.14	3 0 0	2 0 0	2 5 0

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	574.32	257.06	123.94	123.94	172.30	172.30
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	68.92	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	68.92	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	30.87	6.46	56.26	44.91
15	159.30	68.92	46.90	6.46	56.26	44.91
16	159.30	68.92	46.90	6.46	56.26	44.91
50	41.27	8.45	1.63	0.88	15.23	10.15
51	41.27	8.45	1.63	0.88	15.23	10.15
52	41.27	8.45	1.63	0.88	15.23	10.15
53	41.27	8.45	1.63	0.88	15.23	10.15

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	0.077	0.699	0.000	0.037	0.000	0.702	#16	0.524	Not Required	Pass
2	0.004	0.622	0.229	0.126	0.042	0.851	#13	0.054	Not Required	Pass
3	0.001	0.932	0.085	0.093	0.036	0.976	#13	0.046	Not Required	Warn
4	0.001	0.903	0.025	0.090	0.002	0.917	#13	0.082	Not Required	Pass
5	0.001	0.578	0.021	0.093	0.008	0.588	#13	0.076	Not Required	Pass
6	0.001	0.932	0.085	0.093	0.036	0.976	#13	0.046	Not Required	Warn
7	0.001	0.578	0.021	0.093	0.008	0.588	#13	0.076	Not Required	Pass
8	0.015	0.235	0.156	0.056	0.011	0.249	#13	0.535	Not Required	Pass
9	0.017	0.104	0.049	0.001	0.000	0.158	#13	0.137	Not Required	Pass
10	0.001	0.903	0.025	0.090	0.002	0.917	#13	0.082	Not Required	Pass
11	0.007	0.242	0.156	0.058	0.011	0.250	#13	0.357	Not Required	Pass
12	0.004	0.622	0.229	0.126	0.042	0.851	#13	0.054	Not Required	Pass
13	0.007	0.590	0.030	0.072	0.005	0.599	#13	0.204	Not Required	Pass
14	0.010	0.580	0.021	0.070	0.005	0.585	#13	0.306	Not Required	Pass
15	0.007	0.242	0.156	0.058	0.011	0.250	#13	0.357	Not Required	Pass
16	0.015	0.235	0.156	0.056	0.011	0.249	#13	0.357	Not Required	Pass
50	0.170	0.010	0.001	0.002	0.001	0.095	#21	0.783	Not Required	Pass
51	0.034	0.007	0.013	0.002	0.002	0.033	#21	0.522	Not Required	Pass
52	0.170	0.010	0.001	0.002	0.001	0.095	#24	0.783	Not Required	Pass
53	0.034	0.007	0.013	0.002	0.002	0.033	#24	0.522	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

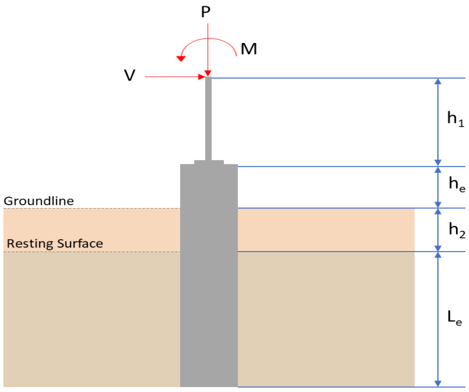
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48 \text{ in}$ - Pile width
$D = 48 \text{ in}$ - Pile depth
$L = 7.75 \text{ ft}$ - 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)	12.726	19.668
V_x (kip)	-3.778	-6.296
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.001
M_z (kipft)	51.098	86.580

Material Properties

$f'_{ck} = 2.5 \text{ ksi}$ - 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,

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

$$H_o = \frac{(-3.778 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60159 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(51.098\ \text{kipft}) + ((-3.778\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 8.1366\ \text{kipft/ft}$$

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:
$L_{e,x} = 7.2919\ \text{ft}$ - 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,

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

$$L_{e,req} = MAX[(7.2919\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 7.292\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 7.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.292\ \text{ft})}{(7.75\ \text{ft})}$$

$$Ratio = 0.9409$$

Status: PASS
Ratio: 0.940

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(12.726\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.79538\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.79538\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.39769$$

Status: PASS
Ratio: 0.400

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.9375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.60159\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 8.1366\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.1366\ \mathrm{kipft/ft}) \times (7.75\ \mathrm{ft})) + \left(3 \times (-0.60159\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft})^2\right)}{(6 \times (8.1366\ \mathrm{kipft/ft})) + (4 \times (-0.60159\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))}$$

$$a = 5.3452\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.1366\ \mathrm{kipft/ft})) + (3 \times (-0.60159\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]^2}{(7.75\ \mathrm{ft})^2 \times [(3 \times (8.1366\ \mathrm{kipft/ft})) + (2 \times (-0.60159\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]}$$

$$p = 0.28513\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (8.1366\ \mathrm{kipft/ft})) + ((-0.60159\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]}{(7.75\ \mathrm{ft})^2}$$

$$s = 1.1599\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.3452\ \mathrm{ft})}{2}$$

$$p_a = 0.40089\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.28513\ \mathrm{kip/ft^2}\right)}{\left(0.40089\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.71124$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.75\ \mathrm{ft})$$

$$p_s = 1.1625\ \mathrm{kip/ft^2}$$

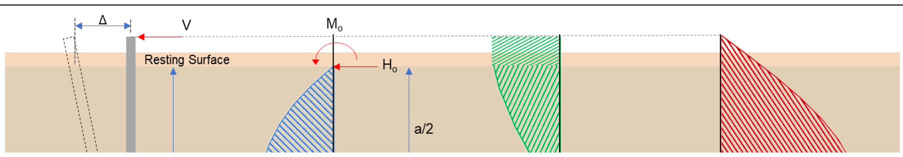
$Ratio$ - Lateral soil capacity

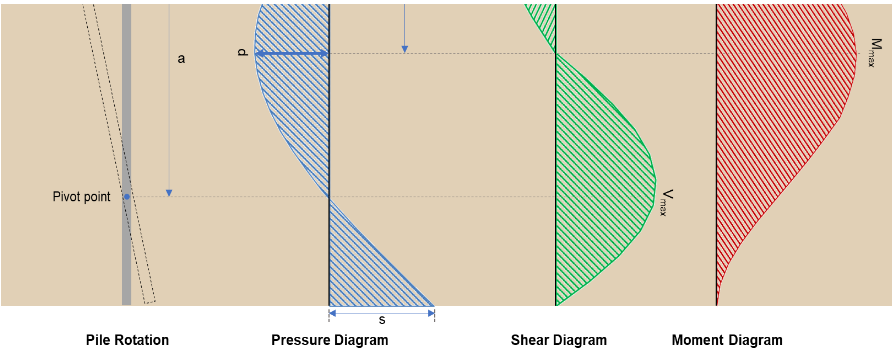
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.1599\ \mathrm{kip/ft^2}\right)}{\left(1.1625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.99775$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.296\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -1.0025\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(86.58\ \mathrm{kipft}) + ((-6.296\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 13.787\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(13.787\ \mathrm{kipft/ft})}{(-1.0025\ \mathrm{kip/ft})}$$

$$E = 13.752\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.787\ \mathrm{kipft/ft}) \times (7.75\ \mathrm{ft})) + \left(3 \times (-1.0025\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft})^2\right)}{(6 \times (13.787\ \mathrm{kipft/ft})) + (4 \times (-1.0025\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))}$$

$$a = 5.343\ \mathrm{ft}$$

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

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

$$V_{max} = ((-1.0025\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.752\ \mathrm{ft})}{(7.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.343\ \mathrm{ft})}{(7.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (13.752\ \mathrm{ft})}{(7.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.343\ \mathrm{ft})}{(7.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 15.237\ \mathrm{kip}$$

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

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

$$M_{max} = ((-1.0025\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (7.75\ \mathrm{ft})) \times \left[\left(\frac{(13.752\ \mathrm{ft})}{(7.75\ \mathrm{ft})} + \frac{(5.343\ \mathrm{ft})}{2 \times (7.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (13.752\ \mathrm{ft})}{(7.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.343\ \mathrm{ft})}{2 \times (7.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (13.752\ \mathrm{ft})}{(7.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.343\ \mathrm{ft})}{2 \times (7.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 56.219 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\, b}$$

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00015924 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (7.75 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7.75 \text{ ft})^2\right)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7.75 \text{ ft}))}$$

$$a = 5.1667 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o\, b}{L_e}\right) \left(\frac{a}{L_e} - 1\right) \left(\frac{a}{L_e}\right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(7.75 \text{ ft})}\right) \times \left(\frac{(5.1667 \text{ ft})}{(7.75 \text{ ft})} - 1\right) \times \left(\frac{(5.1667 \text{ ft})}{(7.75 \text{ ft})}\right)^2$$

$$V_{max} = 0.00014611 \text{ kip}$$

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

$$M_{max} = (M_o\, b) \left[1 - \left(4\, \frac{a}{2\, L_e}^3\right) + \left(3\, \frac{a}{2\, L_e}^4\right)\right]$$

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(5.1667 \text{ ft})}{2 \times (7.75 \text{ ft})}^3\right) + \left(3 \times \frac{(5.1667 \text{ ft})}{2 \times (7.75 \text{ ft})}^4\right)\right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(19.668 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -83.943 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-83.943 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(4.1472\ \mathrm{in}^2\right)}{\left(0.3068\ \mathrm{in}^2\right)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (14) \times \frac{\pi \times (0.625\ \mathrm{in})^2}{4}$$

$$A_{st} = 4.2951\ \mathrm{in}^2$$

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(4.1472\ \mathrm{in}^2\right)}{\left(4.2951\ \mathrm{in}^2\right)}$$

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(19.668\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.007352$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

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

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

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 $f'_{ck} = 2.5 \text{ ksi} \to 2500 \text{ psi}$,

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 $f'_{ck} = 2.5 \text{ ksi} \to 2500 \text{ psi}$, $P = 19.668 \text{ kip} \to 19668 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(19668 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.11 \text{ kip}$$

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

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (121.11 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 121.11 \text{ kip}$$

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

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

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

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

$$V_s = MIN\left[(757.28\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((121.11\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 111.8\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 15.237\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(15.237\ \mathrm{kip})}{(111.8\ \mathrm{kip})}$$

$$Ratio = 0.13628$$

Status: **PASS**
Ratio: **0.140**

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:
$\phi M_{n,1}$

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

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

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

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

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

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

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

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:

$M_{max} = 56.219\ \mathrm{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(56.219\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$

$$Ratio = 0.22524$$

Status: **PASS**
Ratio: **0.230**

Considering z-direction:

$M_{max} = 0.00056617\ \mathrm{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	ϕM_n $Ratio = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 2.2683 \times 10^{-6}$	Status: **PASS** Ratio: **0.000**