Your Project Calculations

Project Name: MARCI Cleaver - Round - V1Jb

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MARCI%20Cleaver%20-%20Round%20-%20V1Jb&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=VpR3RwfLq5vjLIR1Uzq6wUc4Hd6PJ8vnI7d1bXLoFqvwpo2chE0KyLbufHPHVZW2

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-XD-57-L-4Hx3W-STRUTS-BG2D
Duty Classification:	XD
Module Width:	40.50 in
Module Length:	73.40in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Desired Tilt Angle:	30
Front Edge Clearance:	8
Total Array Height at Tilt:	14.79 ft
Total Frame Length:	17.00 ft
Frame Weight:	1077 lbs
Array Dimensions N/S:	13.67 ft
Array Dimensions E/W:	18.60 ft
Rail Length:	164.00 in
Rail Spacing:	3.06 ft
Rail Check:	Not Checked

Support Specifications

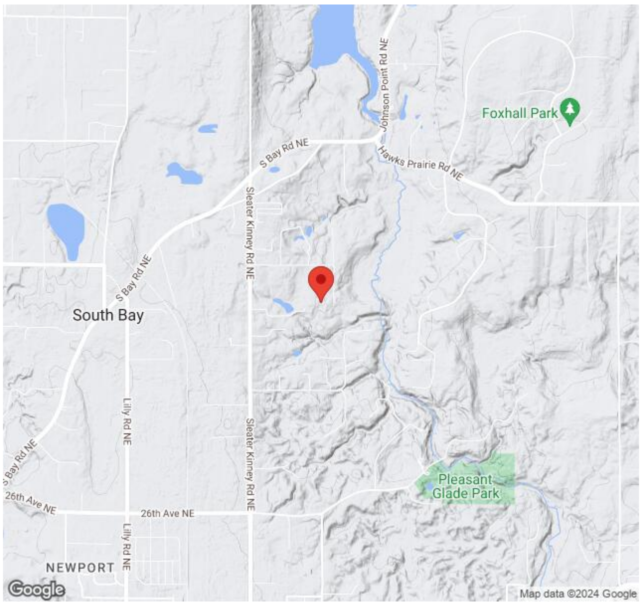
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.42 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø30 in
Foundation Depth (below grade):	Pile 1: 15.00 ft
Foundation Volume:	2.727 y^3
Foundation Result:	FAILED Try increasing foundation size and/or type and re-running foundation design check on right panel.
Mount Twist:	0.001205 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	3415 Quail Creek Ln NE, Olympia, WA 98516, USA
Wind Speed:	100 mph
Snow Load:	40 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.040000 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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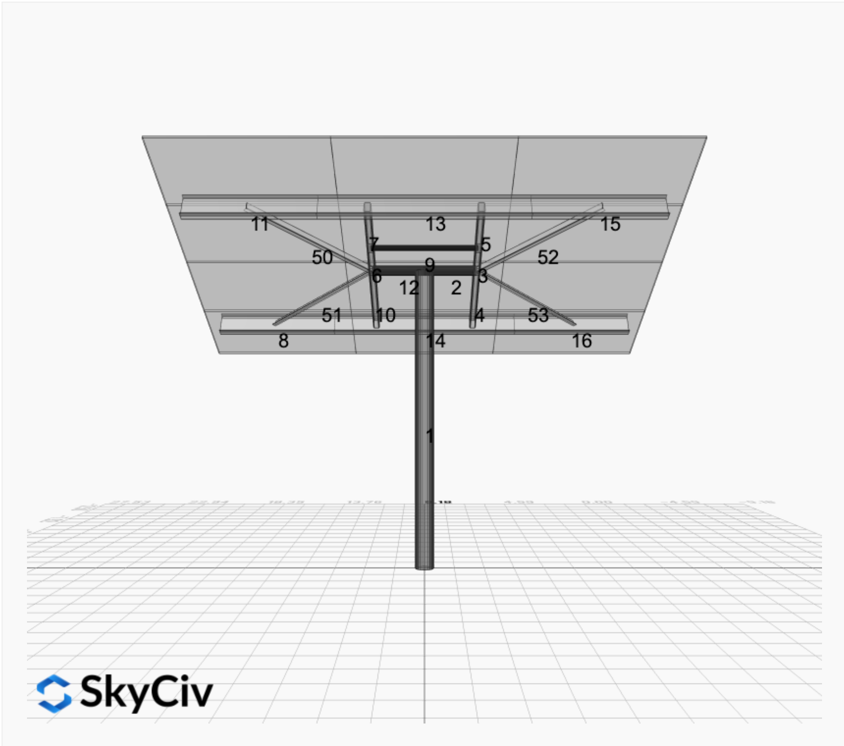
Design Notes:

- 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
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)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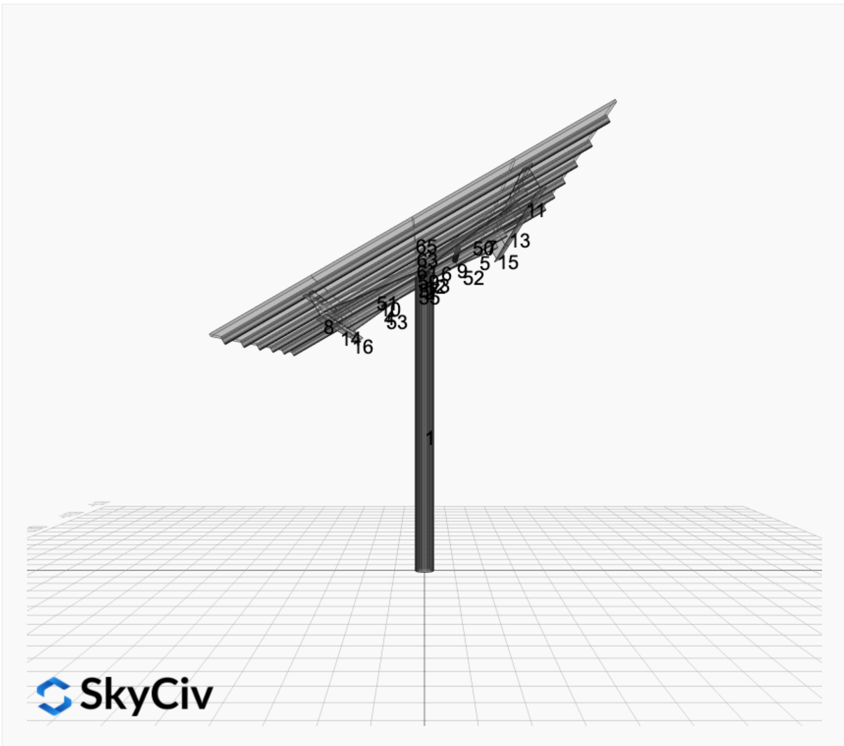
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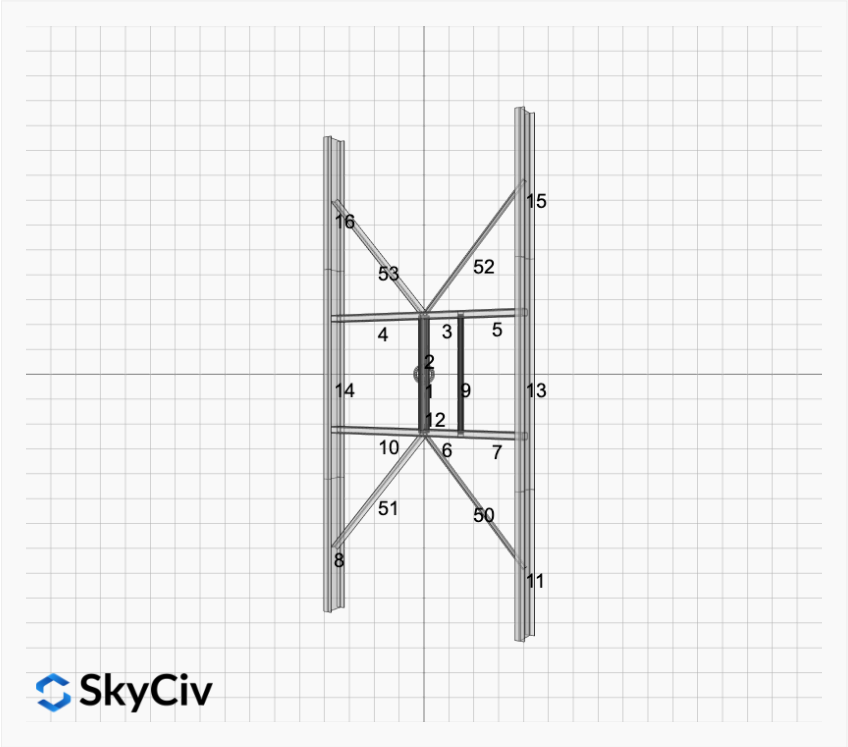
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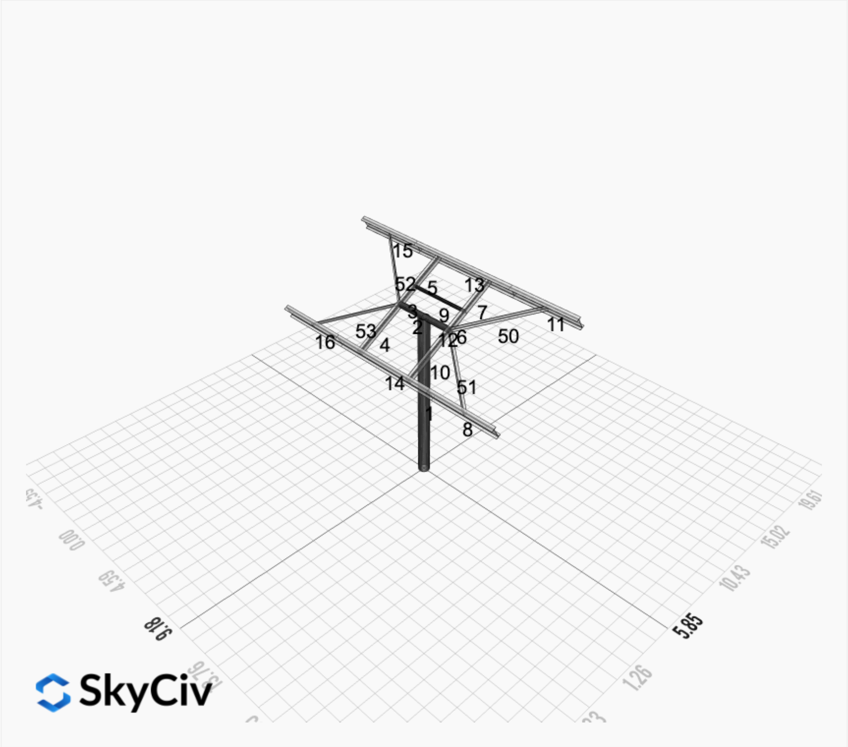
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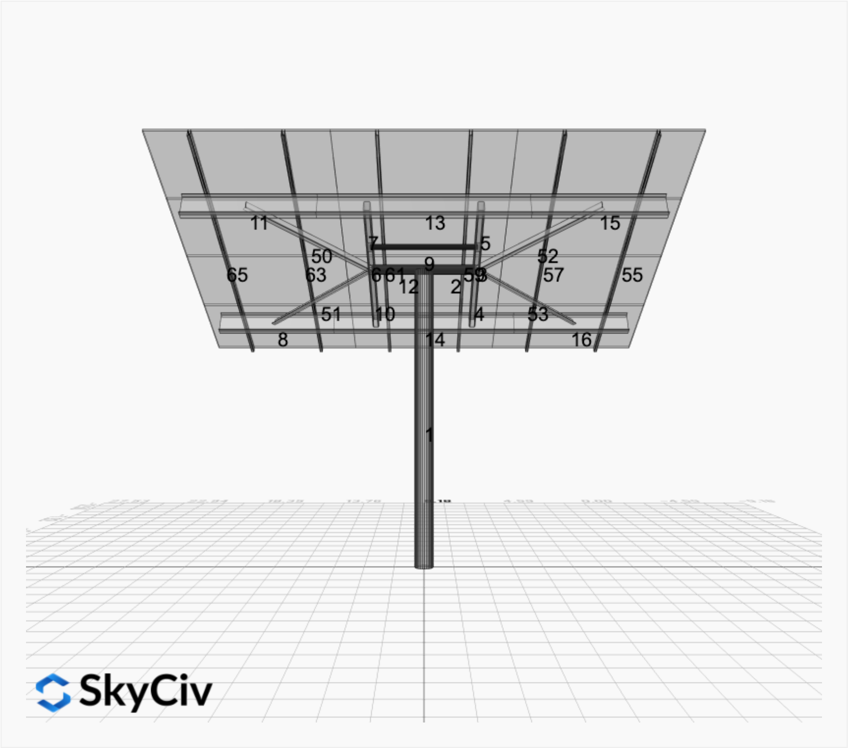
10.43
15.02
19.61
1.26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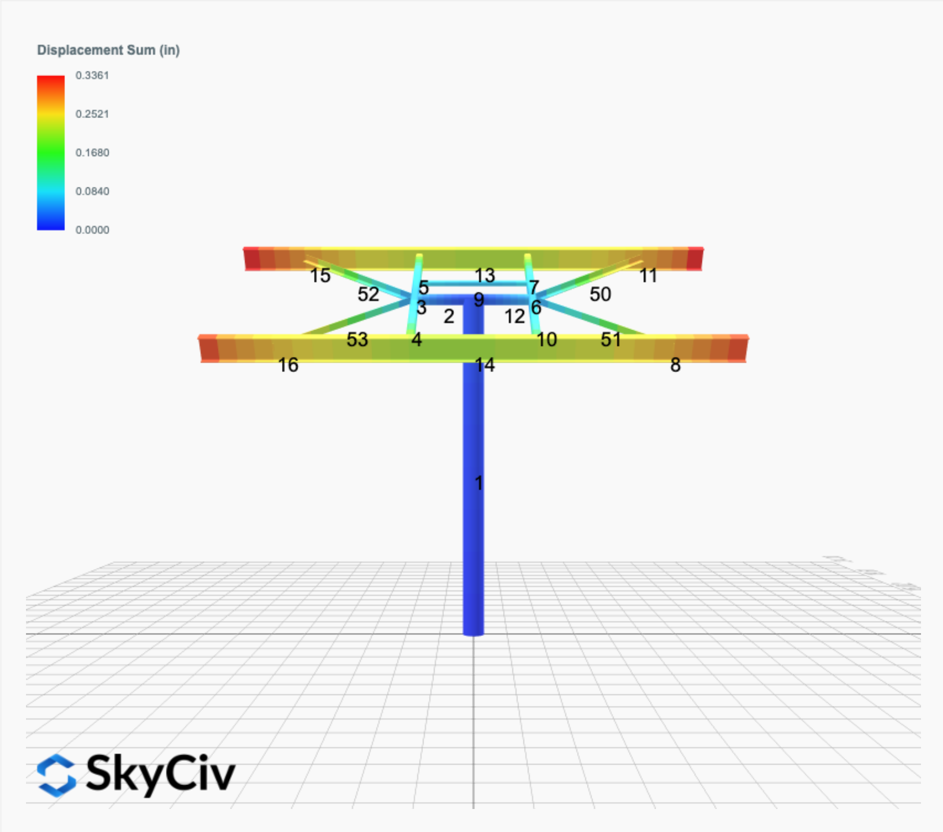
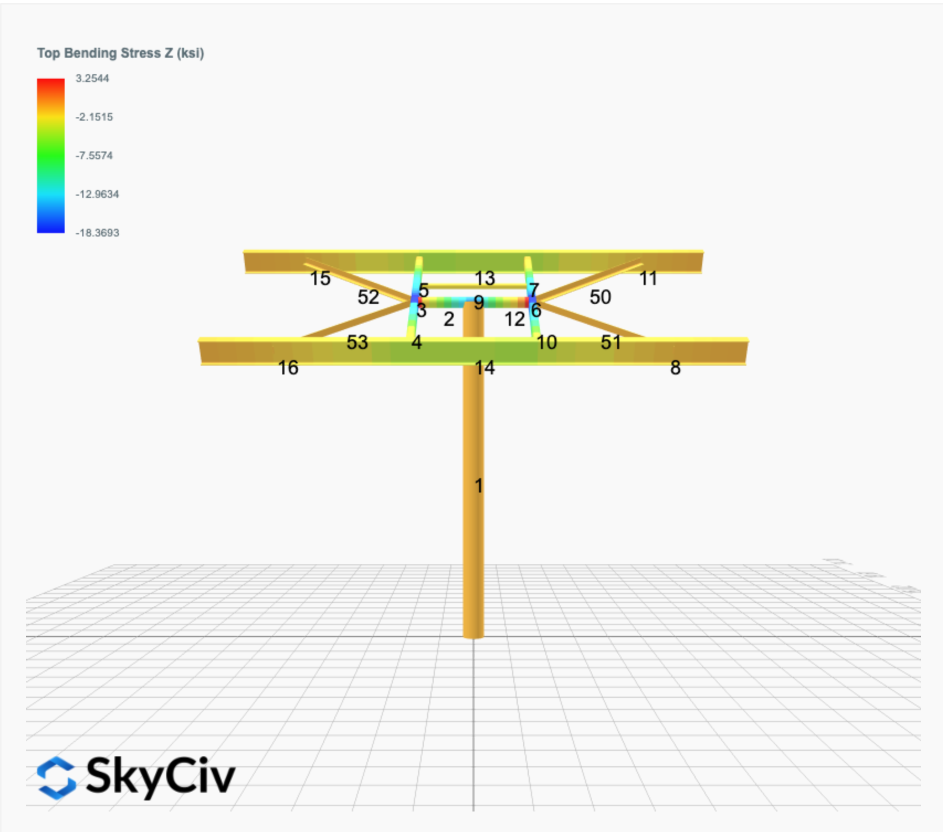
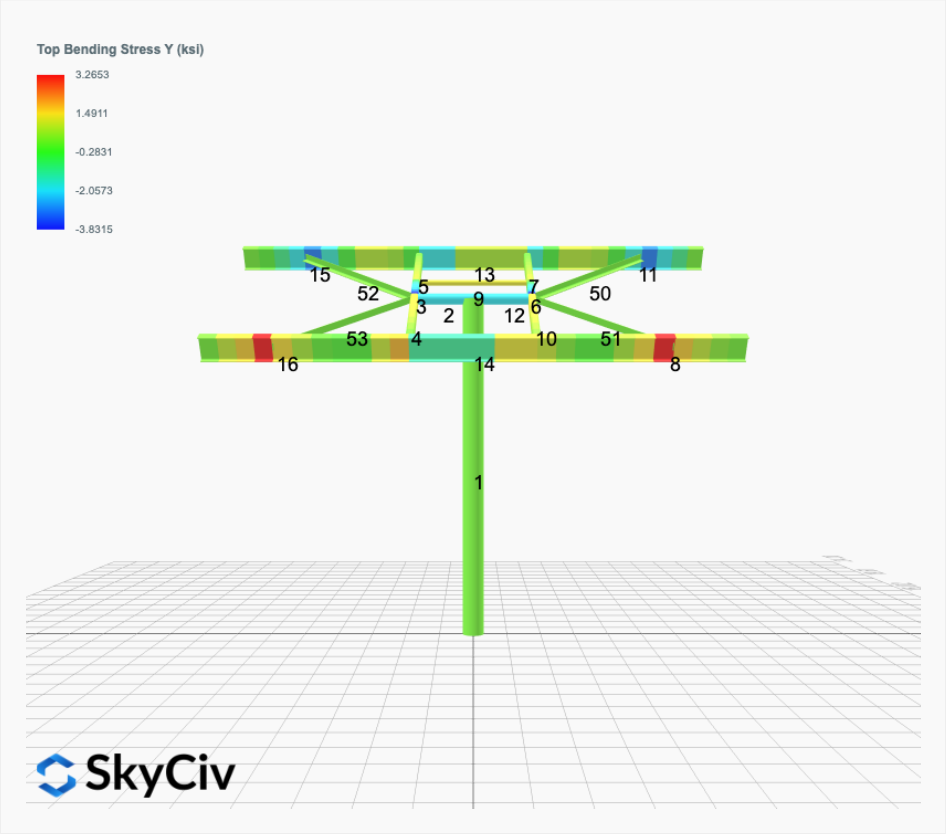
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
3.2653
1.4911
-0.2831
-2.0573
-3.8315
15
13
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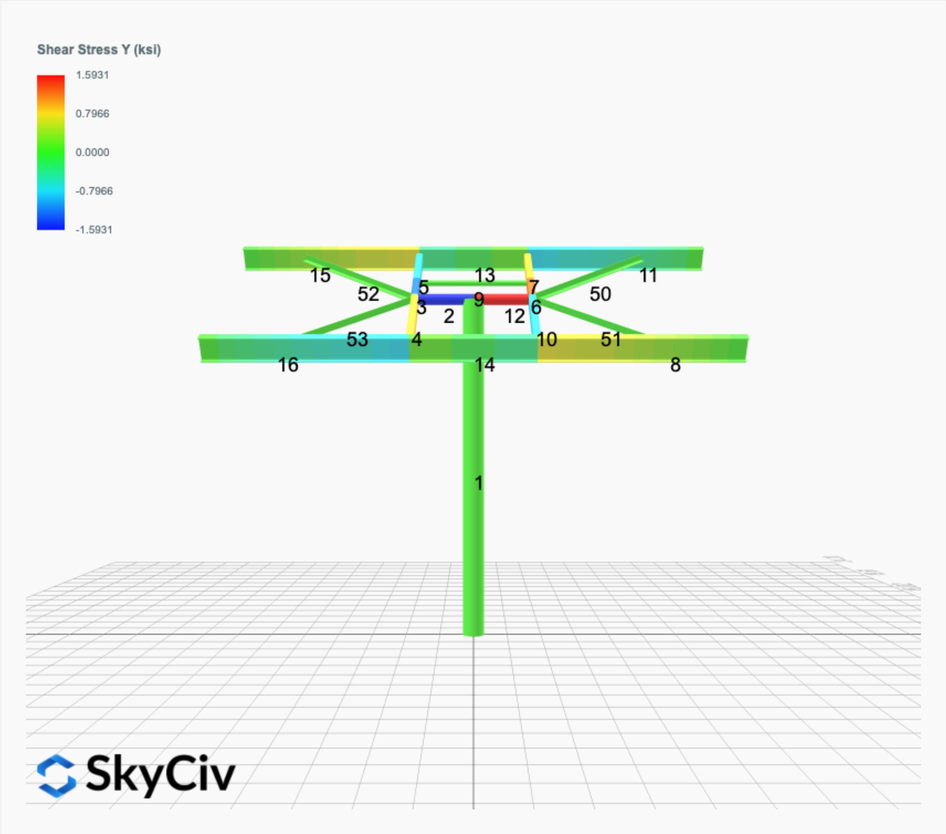
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
1.5931
0.7966
0.0000
-0.7966
-1.5931
15
13
11
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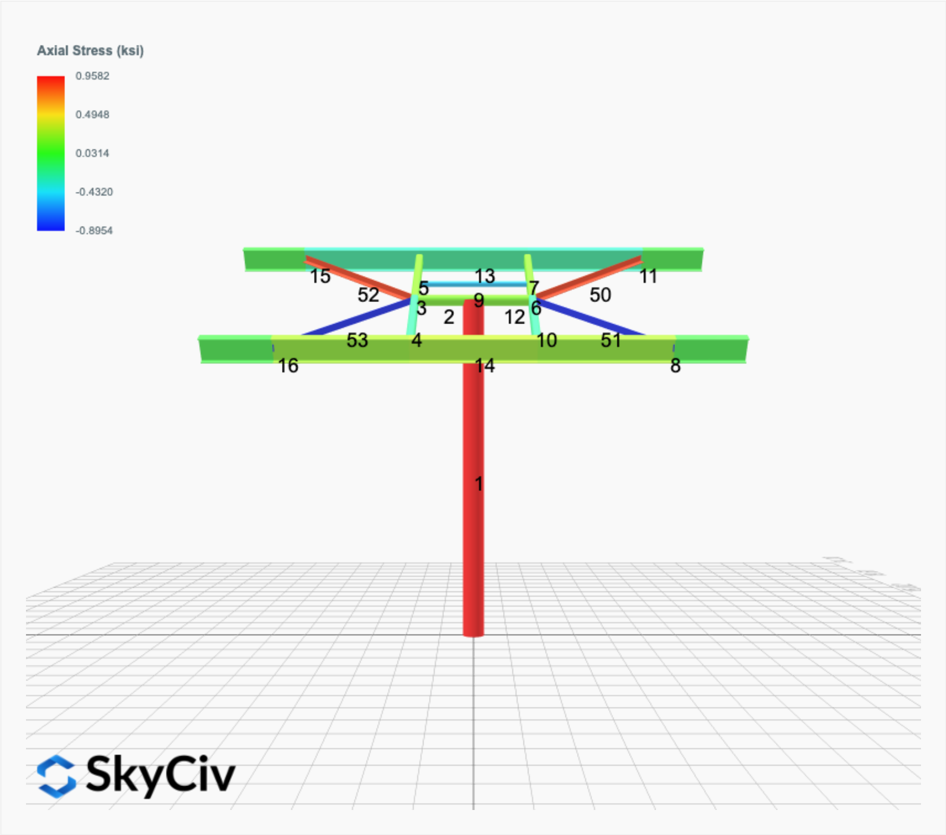
SkyCiv


Axial Stress (ksi)
0.9582
0.4948
0.0314
-0.4320
-0.8954
15
52
5
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50
3
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2
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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	2.2740	0.0000	-0.0000	-0.0000	0.0298
ULS: 2. D + L	0.0000	2.2740	0.0000	-0.0000	-0.0000	0.0298
ULS: 3. D + (S or Lr or R)	0.0000	10.3223	0.0000	-0.0000	-0.0000	0.1022
ULS: 3. D + (S or Lr or R)	0.0000	2.2740	0.0000	-0.0000	-0.0000	0.0298
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	8.3102	0.0000	-0.0000	-0.0000	0.0841
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.2740	0.0000	-0.0000	-0.0000	0.0298
ULS: 5b. D + 0.7E	0.0000	2.2740	0.0000	-0.0000	-0.0000	0.0298
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	8.3102	0.0000	-0.0000	-0.0000	0.0841
ULS: 8. 0.6D + 0.7E	0.0000	1.3644	0.0000	-0.0000	-0.0000	0.0179
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5069	6.6161	0.0000	-0.0000	-0.0000	29.2830
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5069	6.6161	0.0000	-0.0000	-0.0000	29.2830
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1488	-1.4477	0.0000	-0.0000	-0.0000	-24.0588
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7906	-0.8274	0.0000	-0.0000	-0.0000	-28.8550
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8802	11.5668	0.0000	-0.0000	-0.0000	22.0240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8802	11.5668	0.0000	-0.0000	-0.0000	22.0240
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6116	5.5189	0.0000	-0.0000	-0.0000	-17.9824
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3430	5.9841	0.0000	-0.0000	-0.0000	-21.5795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8802	5.5306	0.0000	-0.0000	-0.0000	21.9697
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8802	5.5306	0.0000	-0.0000	-0.0000	21.9697
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6116	-0.5173	0.0000	-0.0000	-0.0000	-18.0367
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3430	-0.0521	0.0000	-0.0000	-0.0000	-21.6338
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5069	5.7065	0.0000	-0.0000	-0.0000	29.2710
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5069	5.7065	0.0000	-0.0000	-0.0000	29.2710
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1488	-2.3573	0.0000	-0.0000	-0.0000	-24.0708
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7906	-1.7370	0.0000	-0.0000	-0.0000	-28.8669

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	19.2245
Shear X	-4.1782
Shear Z	0.0000
Moment X	0.0008
Moment Y (Twist)	0.0012
Moment Z	49.9374

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	11.5668
Shear X	-2.5069
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	29.2830

Project Details

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

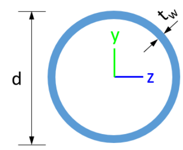
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

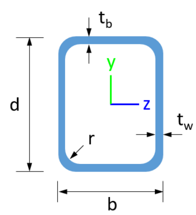
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

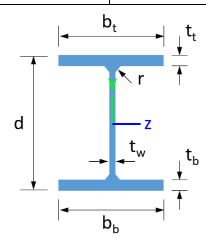
Section Dimensions



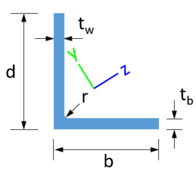
ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



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



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
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	9	23.98	23.98	11.42	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.50,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.72,1.67,1.67,1.66,1.62	300	200	1
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
8	20	4.75	4.75	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.32,2.32,2.31,2.31,2.32,2.32,2.31,2.26,2.32,2.32,2.32,2.31,2.32,2.32,2.30,2.35,2.32,2.32,2.31,2.30	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.68,1.67,1.67,1.66,1.50,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.72,1.67,1.67,1.66,1.62	300	200	1
11	20	4.75	4.75	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.32,2.32,2.31,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.32,2.31,2.32,2.32,2.30,2.31,2.32,2.32,2.31,2.32	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.06,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.03,1.03,1.03	300	200	1
14	20	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.08,1.03,1.03,1.03,1.01,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.24,1.03,1.03,1.03,1.02	300	200	1
15	20	4.75	4.75	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.32,2.32,2.31,2.32,2.32,2.32,2.31,2.31,2.32,2.32,2.32,2.31,2.32,2.32,2.30,2.31,2.32,2.32,2.31,2.32	300	200	1
16	20	4.75	4.75	4.75	2.34,2.32,2.34,2.32,2.33,2.34,2.32,2.32,2.31,2.31,2.32,2.32,2.31,2.26,2.32,2.32,2.32,2.31,2.32,2.32,2.30,2.35,2.32,2.32,2.31,2.30	300	200	1
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

52	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
53	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

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	187.46	83.29	83.29	113.39	113.39
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	105.96	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	105.96	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	31.42	6.46	56.26	44.91
14	159.30	97.43	30.99	6.46	56.26	44.91
15	159.30	105.96	46.90	6.46	56.26	44.91
16	159.30	105.96	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	0.103	0.600	0.000	0.037	0.000	0.637	#13	0.490	Not Required	Pass
2	0.006	0.579	0.161	0.125	0.028	0.709	#21	0.054	Not Required	Pass
3	0.006	0.804	0.136	0.080	0.055	0.943	#21	0.046	Not Required	Pass
4	0.006	0.795	0.055	0.079	0.008	0.854	#21	0.082	Not Required	Pass
5	0.006	0.499	0.022	0.080	0.005	0.524	#21	0.076	Not Required	Pass
6	0.006	0.804	0.136	0.080	0.055	0.943	#21	0.046	Not Required	Pass
7	0.006	0.499	0.022	0.080	0.005	0.524	#21	0.076	Not Required	Pass
8	0.011	0.121	0.085	0.042	0.011	0.133	#21	0.363	Not Required	Pass
9	0.023	0.077	0.050	0.001	0.000	0.138	#21	0.137	Not Required	Pass
10	0.006	0.795	0.055	0.079	0.008	0.854	#21	0.082	Not Required	Pass
11	0.007	0.122	0.085	0.043	0.012	0.142	#21	0.242	Not Required	Pass
12	0.006	0.579	0.161	0.125	0.028	0.709	#21	0.054	Not Required	Pass
13	0.007	0.363	0.047	0.058	0.010	0.413	#21	0.204	Not Required	Pass
14	0.014	0.365	0.055	0.058	0.010	0.406	#21	0.306	Not Required	Pass
15	0.007	0.122	0.085	0.043	0.012	0.142	#21	0.242	Not Required	Pass
16	0.011	0.121	0.085	0.042	0.011	0.133	#21	0.242	Not Required	Pass
50	0.185	0.010	0.000	0.002	0.001	0.101	#21	0.783	Not Required	Pass
51	0.037	0.007	0.014	0.002	0.002	0.034	#21	0.522	Not Required	Pass
52	0.185	0.010	0.000	0.002	0.001	0.101	#24	0.783	Not Required	Pass
53	0.037	0.007	0.014	0.002	0.002	0.034	#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

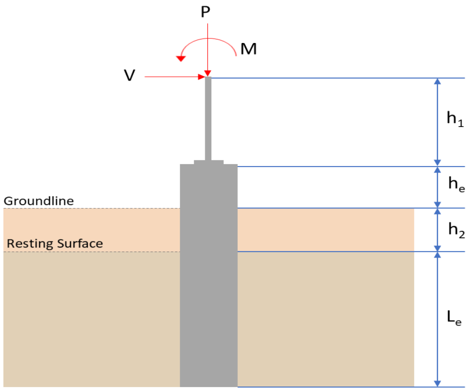
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round
$D = 30\ \mathrm{in}$ - Pile diameter
$L = 15\ \mathrm{ft}$ - 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)	11.567	19.224
V_x (kip)	-2.507	-4.178
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.001
M_z (kipft)	29.283	49.937

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,

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

$$H_o = \frac{(-2.507\ \mathrm{kip})}{(30\ \mathrm{in})}$$

$$H_o = -1.0028\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.283\ \mathrm{kipft}) + ((-2.507\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{(30\ \mathrm{in})}$$

$$M_o = 11.713\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 8.6721\ \mathrm{ft}$ - 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,

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

$$L_{e,req} = MAX[(8.6721\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 8.672\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (15\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 15\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(8.672\ \mathrm{ft})}{(15\ \mathrm{ft})}$$

$$Ratio = 0.57813$$

Status: **PASS**
Ratio: **0.580**

End-bearing Capacity (ASD)

A - Pile cros-section area

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

$$A = \pi \times \left(\frac{(30\ \mathrm{in})}{2}\right)^2$$

$$A = 4.9087\ \mathrm{ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.567\ \mathrm{kip})}{(4.9087\ \mathrm{ft}^2)}$$

$$q = 2.3564\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(2.3564\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 1.1782$$

Status: **FAIL**
Ratio: **1.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(15\ \mathrm{ft})}{(30\ \mathrm{in})}$$

$$L/D = 6$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -1.0028\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 11.713\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.713\ \mathrm{kipft/ft}) \times (15\ \mathrm{ft})) + \left(3 \times (-1.0028\ \mathrm{kip/ft}) \times (15\ \mathrm{ft})^2\right)}{(6 \times (11.713\ \mathrm{kipft/ft})) + (4 \times (-1.0028\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))}$$

$$a = 10.577\ \mathrm{ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (11.713\ \mathrm{kipft/ft})) + (3 \times (-1.0028\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]^{\ 2}}{(15\ \mathrm{ft})^2 \times [(3 \times (11.713\ \mathrm{kipft/ft})) + (2 \times (-1.0028\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]}$$

$$p = 0.003088\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{9.425 \times [(2 \times (11.713\ \mathrm{kipft/ft})) + ((-1.0028\ \mathrm{kip/ft}) \times (15\ \mathrm{ft}))]}{(15\ \mathrm{ft})^2}$$

$$s = 0.35121\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(10.577\ \mathrm{ft})}{2}$$

$$p_a = 0.79324\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.003088\ \mathrm{kip/ft^2}\right)}{\left(0.79324\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0038929$$

Status: **PASS**
Ratio: **0.000**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\ L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (15\ \mathrm{ft})$$

$$p_s = 2.25\ \mathrm{kip/ft^2}$$

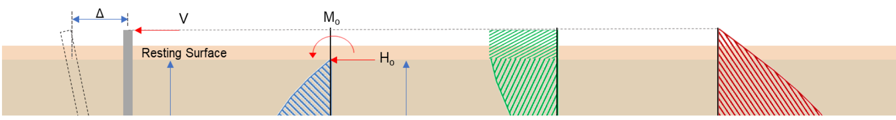
$Ratio$ - Lateral soil capacity

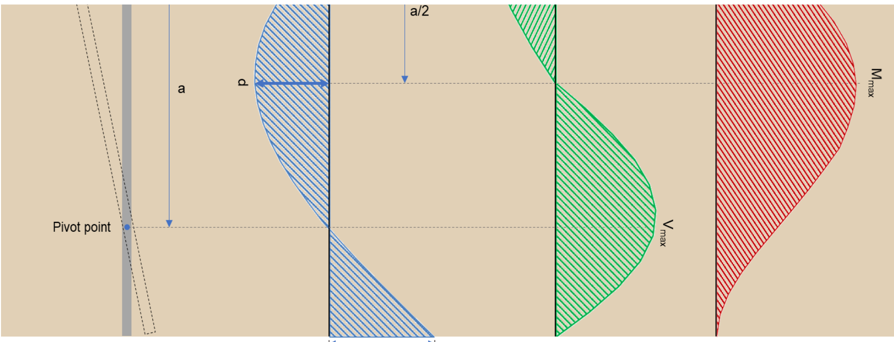
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(0.35121\ \mathrm{kip/ft^2}\right)}{\left(2.25\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.15609$$

Status: **PASS**
Ratio: **0.160**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.178 \text{ kip})}{(30 \text{ in})}$$

$$H_o = -1.6712 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.937 \text{ kipft}) + ((-4.178 \text{ kip}) \times (0 \text{ ft}))}{(30 \text{ in})}$$

$$M_o = 19.975 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(19.975 \text{ kipft/ft})}{(-1.6712 \text{ kip/ft})}$$

$$E = 11.952 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (19.975 \text{ kipft/ft}) \times (15 \text{ ft})) + \left(3 \times (-1.6712 \text{ kip/ft}) \times (15 \text{ ft})^2\right)}{(6 \times (19.975 \text{ kipft/ft})) + (4 \times (-1.6712 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10.569 \text{ ft}$$

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

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

$$V_{max} = ((-1.6712 \text{ kip/ft}) \times (30 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.952 \text{ ft})}{(15 \text{ ft})} + 3\right) \times \left(\frac{(10.569 \text{ ft})}{(15 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.952 \text{ ft})}{(15 \text{ ft})} + 2\right) \times \left(\frac{(10.569 \text{ ft})}{(15 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 8.6568 \text{ kip}$$

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

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

$$M_{max} = ((-1.6712 \text{ kip/ft}) \times (30 \text{ in}) \times (15 \text{ ft})) \times \left[\left(\frac{(11.952 \text{ ft})}{(15 \text{ ft})} + \frac{(10.569 \text{ ft})}{2 \times (15 \text{ ft})}\right) - \left[\left(\frac{4 \times (11.952 \text{ ft})}{(15 \text{ ft})} + 3\right) \times \left(\frac{(10.569 \text{ ft})}{2 \times (15 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.952 \text{ ft})}{(15 \text{ ft})} + 2\right) \times \left(\frac{(10.569 \text{ ft})}{2 \times (15 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 59.3 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{(30 \text{ in})}$$

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{(30 \text{ in})}$$

$$M_o = 0.0004 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0004 \text{ kipft/ft}) \times (15 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (15 \text{ ft})^2\right)}{(6 \times (0.0004 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (15 \text{ ft}))}$$

$$a = 10 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.0004 \text{ kipft/ft}) \times (30 \text{ in})}{(15 \text{ ft})}\right) \times \left(\frac{(10 \text{ ft})}{(15 \text{ ft})} - 1\right) \times \left(\frac{(10 \text{ ft})}{(15 \text{ ft})}\right)^2$$

$$V_{max} = 0.00011852 \text{ kip}$$

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

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

$$M_{max} = ((0.0004 \text{ kipft/ft}) \times (30 \text{ in})) \times \left[1 - \left(4 \times \frac{(10 \text{ ft})}{2 \times (15 \text{ ft})}^3\right) + \left(3 \times \frac{(10 \text{ ft})}{2 \times (15 \text{ ft})}^4\right)\right]$$

$$M_{max} = 0.00088889 \text{ kipft}$$

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 $\alpha = 0.85$ - Alpha factor for axial strength,
$A_g = 706.86 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

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

$$A_{st,required} = Min\left[\frac{\frac{(19.224 \text{ kip})}{(0.65) \times (0.85)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(706.86 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(706.86 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -25.353 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-25.353 \text{ in}^2\right), \left(0.0018 \times \left(706.86 \text{ in}^2\right)\right)\right]$$

$$A_{min} = 1.2723 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{\left(1.2723\ \mathrm{in}^2\right)}{\left(0.3068\ \mathrm{in}^2\right)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(1.2723\ \mathrm{in}^2\right)}{\left(1.8408\ \mathrm{in}^2\right)}$$

$$Ratio = 0.6912$$

Status: **PASS**
Ratio: **0.690**

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 center-to-center spacing of ties,

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

$$s_{ties} = Min\left[\left(16 \times (0.625\ \mathrm{in})\right), \left(48 \times (0.375\ \mathrm{in})\right), (30\ \mathrm{in})\right]$$

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

Summary:

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(706.86\ \mathrm{in}^2\right) - \left(1.8408\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(1.8408\ \mathrm{in}^2\right)\right)\right]$$

$$\phi P_N = 888.76\ \mathrm{kip}$$

$Ratio$ - Capacity

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

$$Ratio = \frac{(19.224\ \mathrm{kip})}{(888.76\ \mathrm{kip})}$$

$$Ratio = 0.02163$$

Status: **PASS**
Ratio: **0.020**

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 30\ \mathrm{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

$$d = 0.80 \times (30\ \mathrm{in})$$

$$d = 24 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(24 \text{ in})}{10}}}, 1\right]$$

$$\lambda_s = 0.76696$$

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

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

$$V_{c,max} = 5 \times (0.76696) \times \sqrt{(2500 \text{ psi})} \times (30 \text{ in}) \times (24 \text{ in})$$

$$V_{c,max} = 138.05 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 19.224 \text{ kip} \rightarrow 19224 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.76696) \times \sqrt{(2500 \text{ psi})} + \frac{(19224 \text{ lbf})}{6 \times (706.86 \text{ in}^2)}\right] \times (30 \text{ in}) \times (24 \text{ in})$$

$$V_{c,a} = 58.485 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.76696) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (30 \text{ in}) \times (24 \text{ in})$$

$$V_{c,b} = 145.22 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(138.05 \text{ kip}), (58.485 \text{ kip}), (145.22 \text{ kip})\right]$$

$$V_c = 58.485 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (30 \text{ in}) \times (24 \text{ in})$$

$$V_{s,a} = 288 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times \left(0.11045 \text{ in}^2\right) \times (60 \text{ ksi}) \times (24 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 31.809 \text{ kip}$$

V_s - Governing shear strength of steel

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

	$$V_s = MIN\left[(288 \text{ kip}), (31.809 \text{ kip})\right]$$ $$V_s = 31.809 \text{ kip}$$		
22.5.1.1	ϕV_n - Allowable shear strength $$\phi V_n = \phi \ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((58.485 \text{ kip}) + (31.809 \text{ kip}))$$ $$\phi V_n = 58.691 \text{ kip}$$ **Considering x-direction:** $V_{max} = 8.6568 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(8.6568 \text{ kip})}{(58.691 \text{ kip})}$$ $$Ratio = 0.1475$$	Status: **PASS** Ratio: **0.150**	
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{\pi \, D^3}{32}$$ $$S_m = \frac{\pi \times (30 \text{ in})^3}{32}$$ $$S_m = 2650.7 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 2650.719in^3$$ $$\phi M_{n,1} = 35.895kipft$$		
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi \, 0.85 \, f_{ck}' \, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(2650.7 \text{ in}^3\right)$$ $$\phi M_{n,2} = 305.11 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(35.895 \text{ kipft}), (305.11 \text{ kipft})]$$ $$\phi M_n = 35.895 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 59.3 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(59.3 \text{ kipft})}{(35.895 \text{ kipft})}$$ $$Ratio = 1.652$$	Status: **FAIL** Ratio: **1.650**	
	Considering z-direction: $M_{max} = 0.00088889 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity		

	$$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.00088889 \text{ kipft})}{(35.895 \text{ kipft})}$$ $$Ratio = 0.000024763$$	Status: **PASS** Ratio: **0.000**	