

Your Project Calculations



Project Name: JohnZarian

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=JohnZarian&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=WaFYtZNgr0DY7sW8r5VSyGP5pNxFlI8Js0tDROVJKm6g7nlarxGfVMZA6jIhTYs3

Array Specification

Product:	Beam
Unique ID:	1P-0-8TOP-HD-24-L-4Hx2W-J30K
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	4
Number of Columns:	2
Total Number of Modules:	8
Desired Tilt Angle:	30
Front Edge Clearance:	5
Total Array Height at Tilt:	11.89 ft
Total Frame Length:	11.50 ft
Frame Weight:	685 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	12.50 ft
Rail Length:	166.40 in
Rail Spacing:	3.08 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.47 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 7.00 ft
Foundation Volume:	1.833 y ³
Foundation Result:	PASSED
Mount Twist:	0.000001 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2790 Skyline Dr, New Meadows, ID 83654, USA
Wind Speed:	95 mph
Snow Load:	85.98 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.037819 ksf



Design Disclaimer

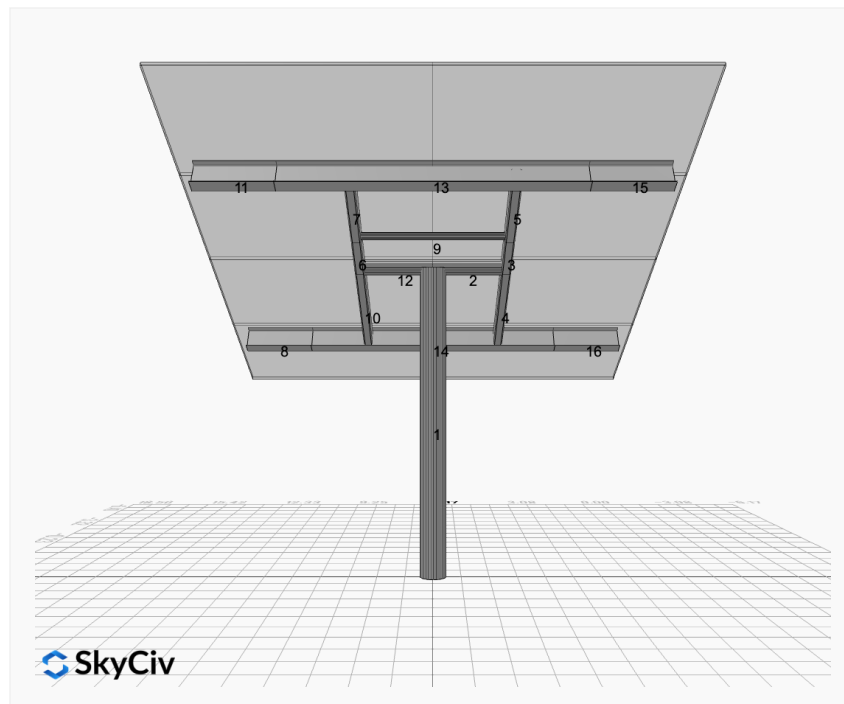
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

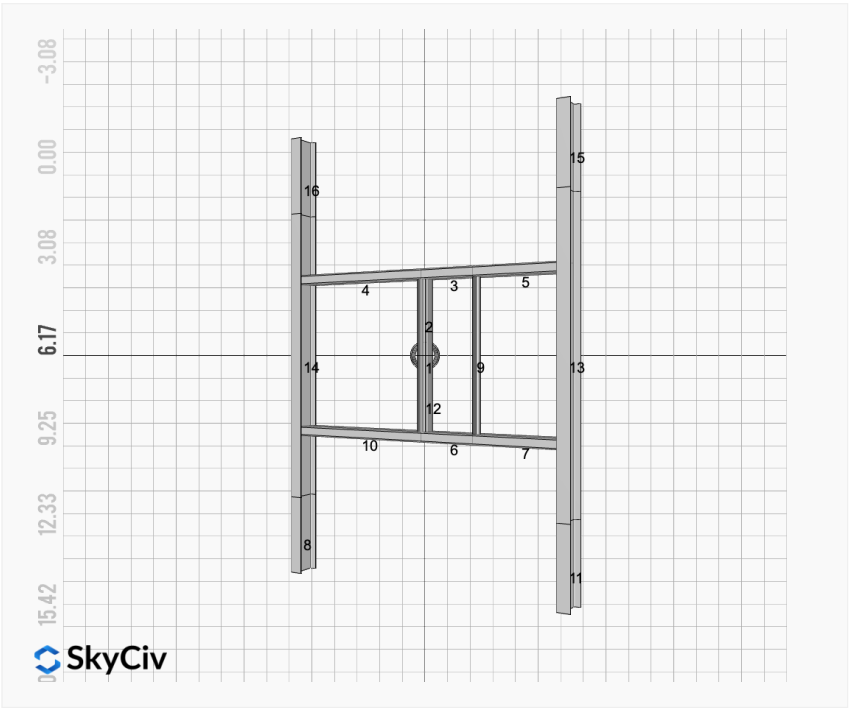
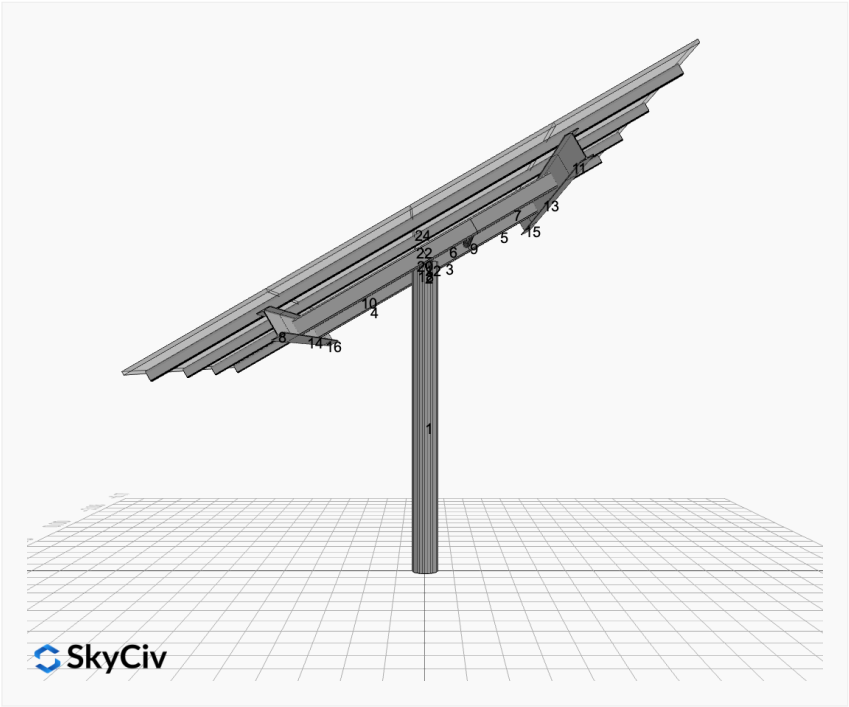
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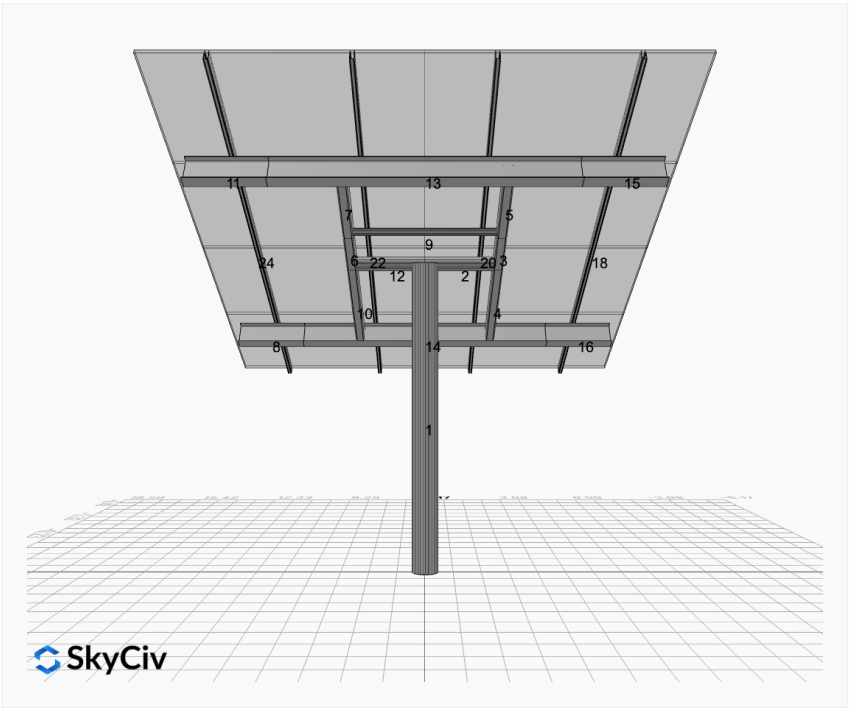
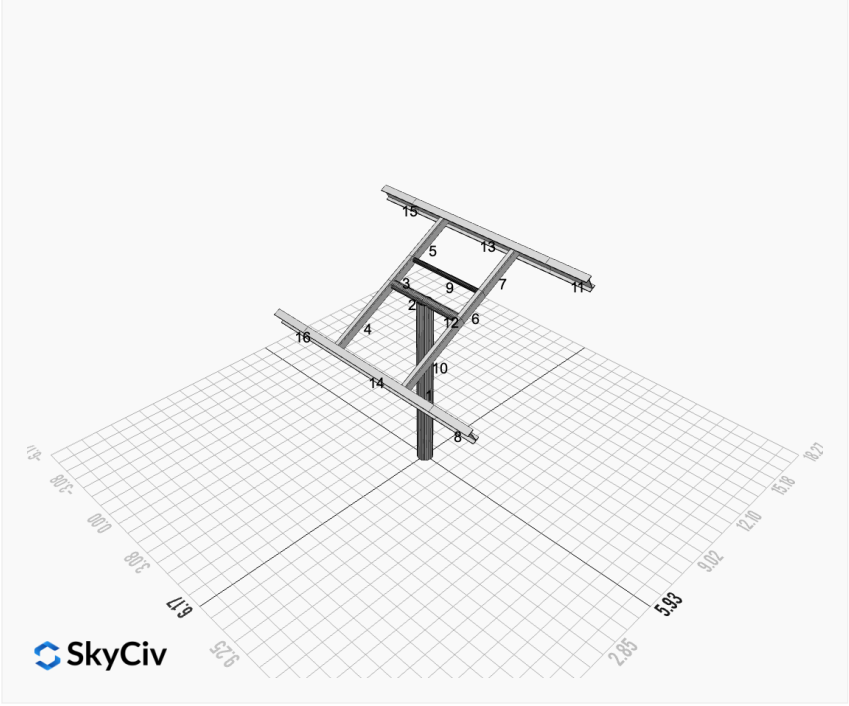
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Design Notes:

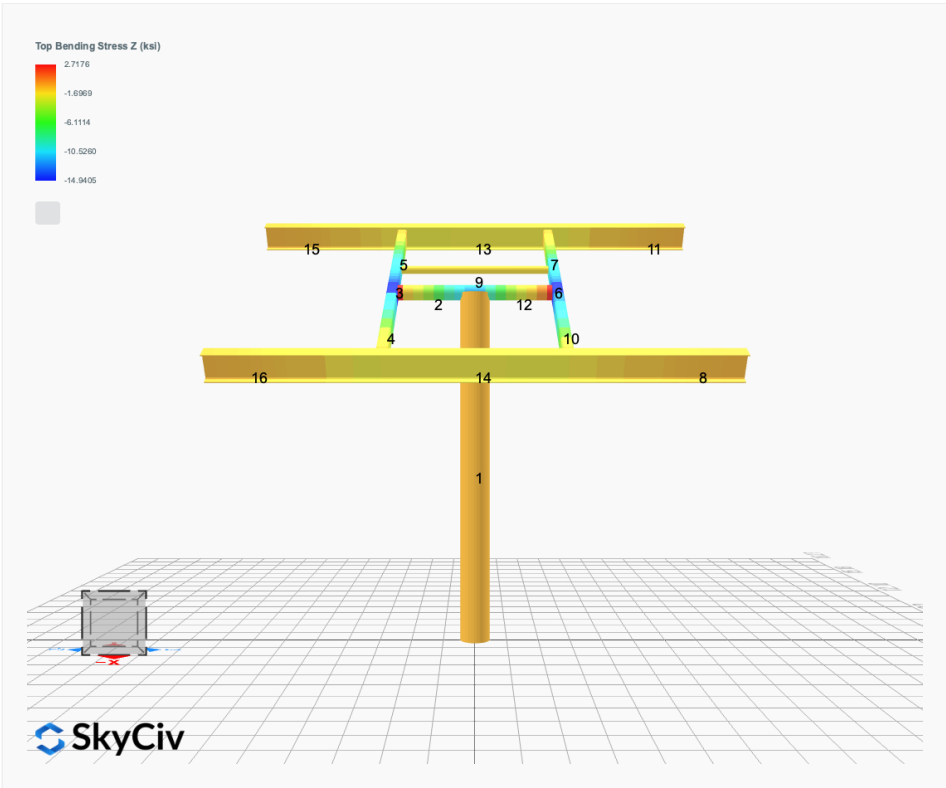
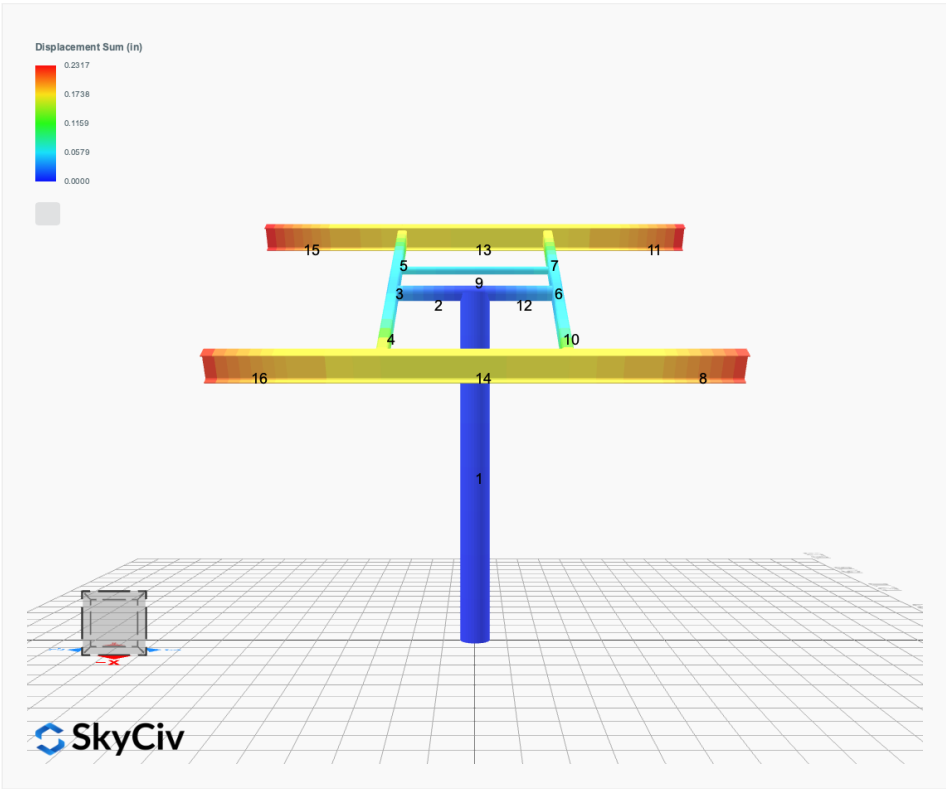
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

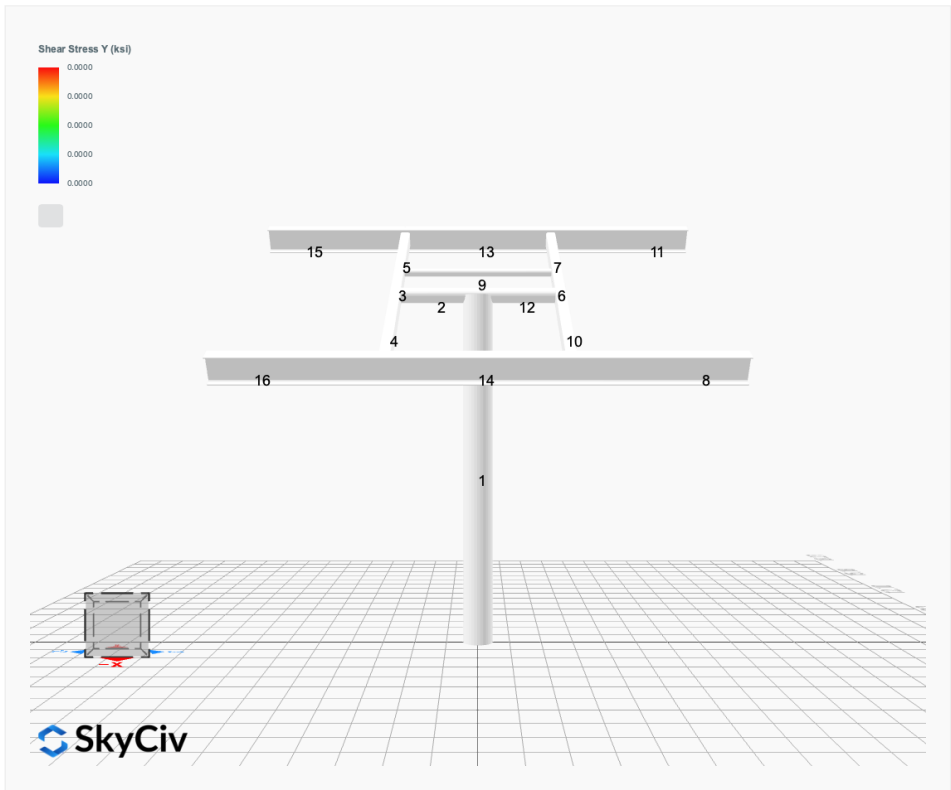
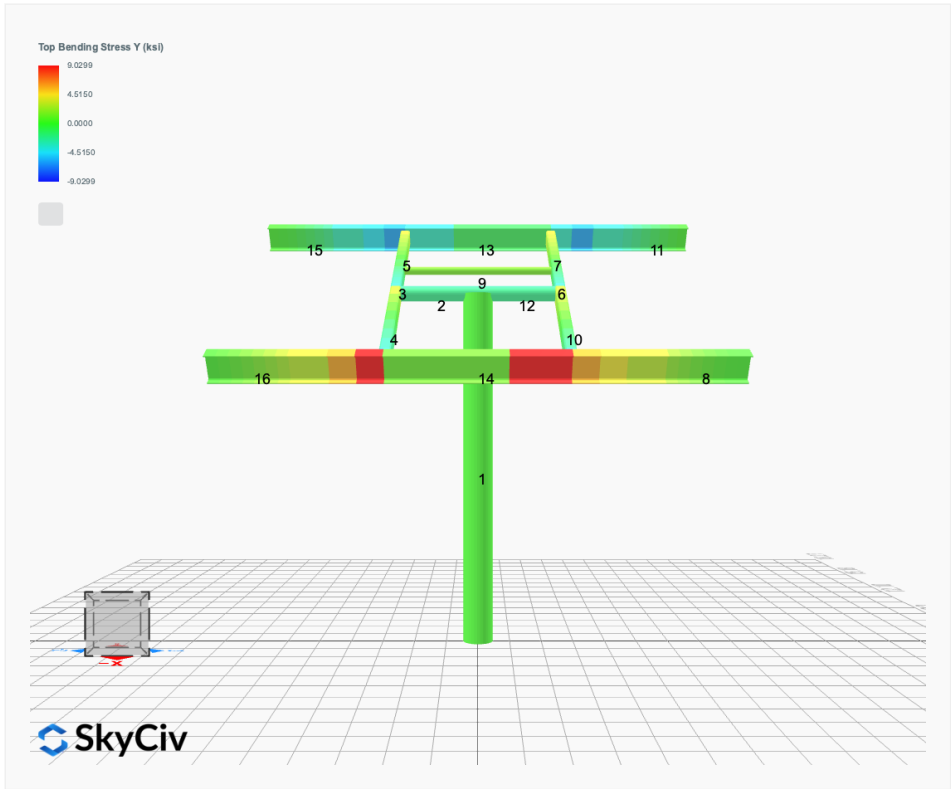


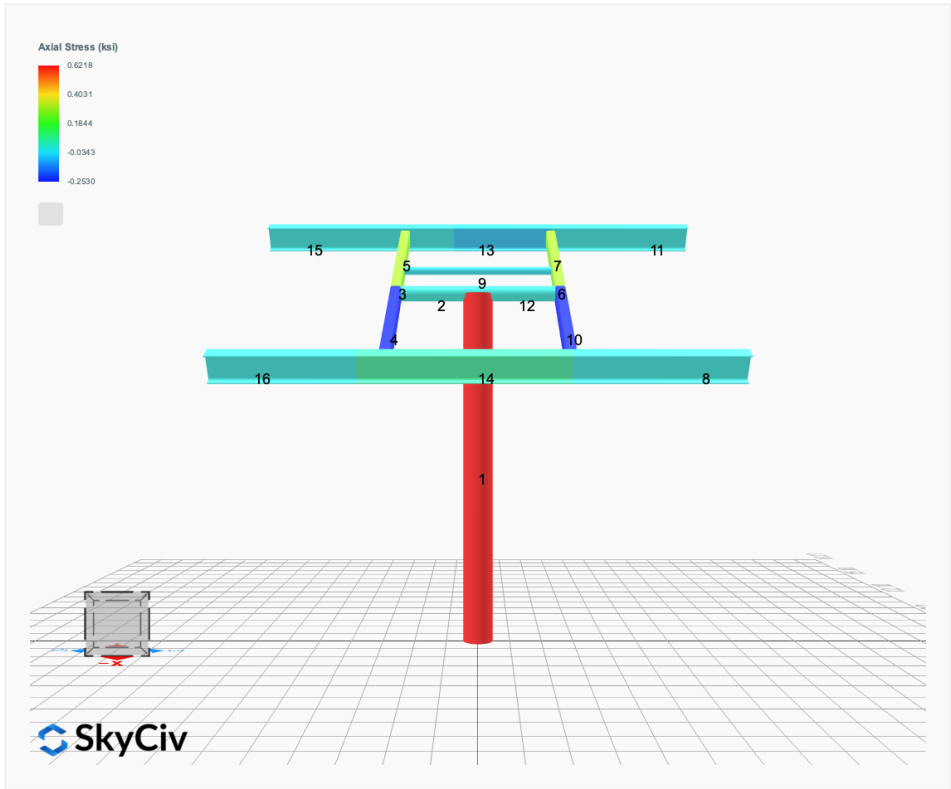




FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.4988	0.0000	0.0000	-0.0000	0.0255
ULS: 2. D + L	-0.0000	1.4988	0.0000	0.0000	-0.0000	0.0255
ULS: 3. D + (S or Lr or R)	-0.0000	6.7216	-0.0000	0.0000	-0.0000	0.0561
ULS: 3. D + (S or Lr or R)	-0.0000	1.4988	0.0000	0.0000	-0.0000	0.0255
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	5.4159	-0.0000	0.0000	-0.0000	0.0485
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.4988	0.0000	0.0000	-0.0000	0.0255
ULS: 5b. D + 0.7E	-0.0000	1.4988	0.0000	0.0000	-0.0000	0.0255
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	5.4159	-0.0000	0.0000	-0.0000	0.0485
ULS: 8. 0.6D + 0.7E	-0.0000	0.8993	0.0000	0.0000	-0.0000	0.0153
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3247	3.7933	-0.0000	0.0000	-0.0000	11.3123
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3247	3.7933	-0.0000	0.0000	-0.0000	11.3123
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1355	-0.4680	0.0000	0.0000	-0.0000	-9.5370
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9462	-0.1402	-0.0000	0.0000	-0.0000	-12.6499
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9935	7.1368	-0.0000	0.0000	-0.0000	8.5136
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9935	7.1368	-0.0000	0.0000	-0.0000	8.5136
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8516	3.9409	-0.0000	0.0000	-0.0000	-7.1234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7097	4.1867	-0.0000	0.0000	-0.0000	-9.4581
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9935	3.2196	-0.0000	0.0000	-0.0000	8.4906
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9935	3.2196	-0.0000	0.0000	-0.0000	8.4906
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8516	0.0237	0.0000	0.0000	-0.0000	-7.1464
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7097	0.2696	0.0000	0.0000	-0.0000	-9.4811
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3247	3.1938	-0.0000	0.0000	-0.0000	11.3021
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3247	3.1938	-0.0000	0.0000	-0.0000	11.3021
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1355	-1.0675	-0.0000	0.0000	-0.0000	-9.5472
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9462	-0.7397	-0.0000	0.0000	-0.0000	-12.6602

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.0672
Shear X	-2.2079
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	21.2766

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	7.1368
Shear X	-1.3247
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	12.6602

Project Details

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

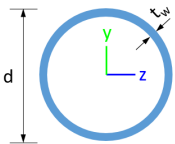
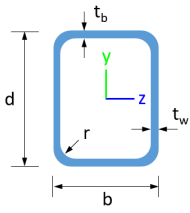
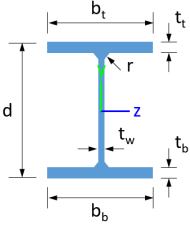
User Name: sales@mtsolar.us
Project Name: JohnZarian
Unit System: imperial



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)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					
								
ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x16	5.00	3.00	0.17	0.17	0.17		
								
ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	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)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D	
1	9	17.78	17.78	8.47	-	300	200	1	
2	5	1.30	1.30	2.00	-	300	200	1	
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.13,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1	
4	16	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.68,1.70,1.67,1.67,1.59,1.68,1.67,1.67,1.65,2.10,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.72,1.67,1.67,1.66,1.55	300	200	1	
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1	
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.13,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1	
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.62,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1	
8	19	4.20	4.20	2.00	2.33,2.33	300	200	1	
9	2	2.60	2.60	4.00	-	300	200	1	
10	16	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.68,1.70,1.67,1.67,1.59,1.68,1.67,1.67,1.65,2.10,1.67,1.67,1.67,1.67,1.68,1.68,1.62,1.72,1.67,1.67,1.66,1.55	300	200	1	
11	19	4.20	4.20	2.00	2.33,2.33	300	200	1	
12	5	1.30	1.30	2.00	-	300	200	1	
13	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.19,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10	300	200	1	
14	19	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.11,1.08,1.08,1.09,1.01,1.08,1.08,1.08,1.09,1.08,1.08,1.09,1.30,1.08,1.08,1.09,1.03	300	200	1	
15	19	4.20	4.20	2.00	2.33,2.33	300	200	1	
16	19	4.20	4.20	2.00	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	377.97	257.02	83.29	83.29	113.39	113.39
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	102.39	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94

10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	102.39	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.75	6.12	40.24	43.62
14	133.20	104.94	23.14	6.12	40.24	43.62
15	133.20	102.39	32.87	6.12	40.24	43.62
16	133.20	102.39	32.87	6.12	40.24	43.62

Design Ratio

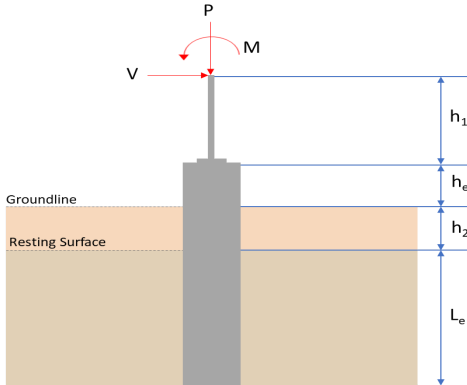
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.047	0.255	0.000	0.019	0.000	0.259	#16	0.363	Not Required	Pass
2	0.000	0.441	0.109	0.099	0.019	0.535	#21	0.053	Not Required	Pass
3	0.011	0.633	0.087	0.064	0.026	0.725	#21	0.045	Not Required	Pass
4	0.011	0.628	0.090	0.063	0.022	0.713	#21	0.080	Not Required	Pass
5	0.011	0.393	0.090	0.063	0.021	0.411	#21	0.074	Not Required	Pass
6	0.011	0.633	0.087	0.064	0.026	0.725	#21	0.045	Not Required	Pass
7	0.011	0.393	0.090	0.063	0.021	0.411	#21	0.074	Not Required	Pass
8	0.000	0.028	0.068	0.023	0.010	0.096	#21	Not Required	Not Required	Pass
9	0.002	0.051	0.038	0.001	0.000	0.090	#21	0.136	Not Required	Pass
10	0.011	0.628	0.090	0.063	0.022	0.713	#21	0.080	Not Required	Pass
11	0.000	0.028	0.068	0.023	0.010	0.096	#21	Not Required	Not Required	Pass
12	0.000	0.441	0.109	0.099	0.019	0.535	#21	0.053	Not Required	Pass
13	0.004	0.149	0.240	0.043	0.018	0.369	#21	0.190	Not Required	Pass
14	0.005	0.153	0.240	0.043	0.018	0.370	#21	0.190	Not Required	Pass
15	0.000	0.028	0.068	0.023	0.010	0.096	#21	Not Required	Not Required	Pass
16	0.000	0.028	0.068	0.023	0.010	0.096	#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/compression)
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

ON
NG

Capacity is provided
Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	SkyCiv Foundation Design Pile Foundation Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial																											
	Pile Input  Geometry Pile shape: round D = 36 in - Pile diameter Le = 7 ft - Total pile length h1 = 0 ft - Lateral load height from the top of the pile, h2 = 0 ft - Depth to resisting surface he = 0 ft - Length of pile above the ground Tabulation of Soil Parameters <table><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></table> Tabulation of Loads <table><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr><tr><td>P (kip)</td><td>7.137</td><td>12.067</td></tr><tr><td>Vx (kip)</td><td>-1.325</td><td>-2.208</td></tr><tr><td>Vz (kip)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mx (kipft)</td><td>0.000</td><td>0.000</td></tr><tr><td>Mz (kipft)</td><td>12.660</td><td>21.277</td></tr></table> Material Properties f'ck = 2.5 ksi - Concrete strength,	Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.137	12.067	Vx (kip)	-1.325	-2.208	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	12.660	21.277	
Layer	Label	Allowable Bearing Pressure (qa) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000																									
Load Component	ASD	LRFD																										
P (kip)	7.137	12.067																										
Vx (kip)	-1.325	-2.208																										
Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	0.000																										
Mz (kipft)	12.660	21.277																										
	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: Ho - Lateral force per length of pile, $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.325\text{ kip})}{(36\text{ in})}$ $H_o = -0.44167\text{ kip/ft}$ Mo - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{D}$																											

	$M_o = \frac{(12.66 \text{ kipft}) + ((-1.325 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 4.22 \text{ kipft/ft}$ 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: $L_{e,x} = 6.4104 \text{ ft}$ - 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, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.4104 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.41 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.41 \text{ ft})}{(7 \text{ ft})}$ $\text{Ratio} = 0.91571$	Status: PASS Ratio: 0.920
	End-bearing Capacity (ASD) A - Pile cross-section area $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$ $A = 7.0686 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_o}{A}$ $q = \frac{(7.137 \text{ kip})}{(7.0686 \text{ ft}^2)}$ $q = 1.0097 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(1.0097 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.50484$	Status: PASS Ratio: 0.500
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$	

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.44167$ kip/ft - Lateral force per length of pile,

$M_o = 4.22$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.22 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.44167 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.22 \text{ kipft/ft})) + (4 \times (-0.44167 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8581 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (4.22 \text{ kipft/ft})) + (3 \times (-0.44167 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^3 \times [(3 \times (4.22 \text{ kipft/ft})) + (2 \times (-0.44167 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.21468 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (4.22 \text{ kipft/ft})) + ((-0.44167 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 1.0287 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8581 \text{ ft})}{2}$$

$$p_a = 0.36436 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21468 \text{ kip/ft}^2)}{(0.36436 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.58921$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

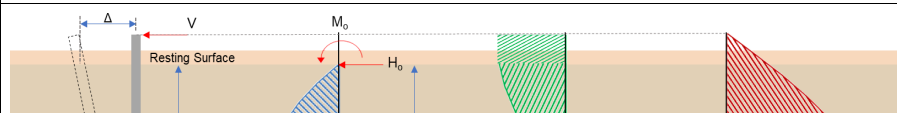
$$\text{Ratio} = \frac{s}{p_s}$$

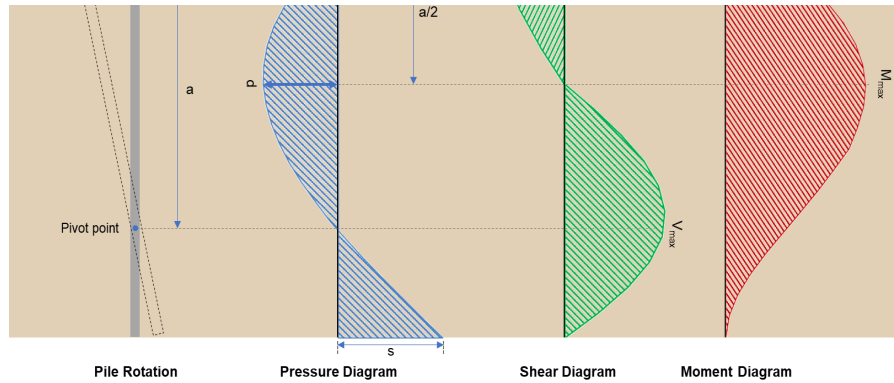
$$\text{Ratio} = \frac{(1.0287 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97975$$

Status: **PASS**
Ratio: **0.590**

Status: **PASS**
Ratio: **0.980**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.208 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.736 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.277 \text{ kipft}) + ((-2.208 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.0923 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.0923 \text{ kipft/ft})}{(-0.736 \text{ kip/ft})}$$

$$E = 9.6363 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0923 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.736 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (7.0923 \text{ kipft/ft})) + (4 \times (-0.736 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.857 \text{ ft}$$

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

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

$$V_{max} = ((-0.736 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.6363 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.857 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.6363 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.857 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.8345 \text{ kip}$$

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

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

$$M_{max} = ((-0.736 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(9.6363 \text{ ft})}{(7 \text{ ft})} + \frac{(4.857 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.6363 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.857 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.6363 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.857 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 22.522 \text{ kipft}$	
Table 22.4.2.1	22.4.2.2, 10.6.1.1	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, $\alpha = 0.85$ - Alpha factor for axial strength, $A_g = 1017.9 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(12.067 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$ $A_{st,required} = -36.996 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-36.996 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$ $A_{min} = 1.8322 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 6$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 1.8408 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$ $\text{Ratio} = 0.99533$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\emptyset$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum center-to-center spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), D]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$ $s_{ties} = 10 \text{ in}$ Summary:	Status: PASS Ratio: 1.000

	Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.85 \left[(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st}) \right]$ $\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$ $\phi P_N = 1253.9 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(12.067 \text{ kip})}{(1253.9 \text{ kip})}$ $Ratio = 0.0096235$	Status: PASS Ratio: 0.010
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 36 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$ $d = 0.80 \times (36 \text{ in})$ $d = 28.8 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.71796$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,max} = 186.09 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.067 \text{ kip} \rightarrow 12067 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(12067 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,a} = 76.486 \text{ kip}$	
22.5.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{c,b} = 204.04 \text{ kip}$ V_c - Governing shear strength of concrete $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(186.09 \text{ kip}), (76.486 \text{ kip}), (204.04 \text{ kip})]$	

22.5.1.2	$V_c = 76.486 \text{ kip}$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a) $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 414.72 \text{ kip}$ 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) $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(414.72 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((76.486 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 74.527 \text{ kip}$ Considering x-direction: $V_{max} = 6.8345 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(6.8345 \text{ kip})}{(74.527 \text{ kip})}$ $Ratio = 0.091705$	Status: PASS Ratio: 0.090
14.5.2.1b	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4580.4 \text{ in}^3$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 62.027 \text{ kipft}$ $\phi M_{n,2}$	

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 22.522 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(22.522 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$\text{Ratio} = 0.3631$$

Status: **PASS**
Ratio: **0.360**