

Project Details



Project Name: Scott Hufford

Date: Tue Sep 16 2025

Location: 11769 Sunset Dr, Parker, CO 80138, USA

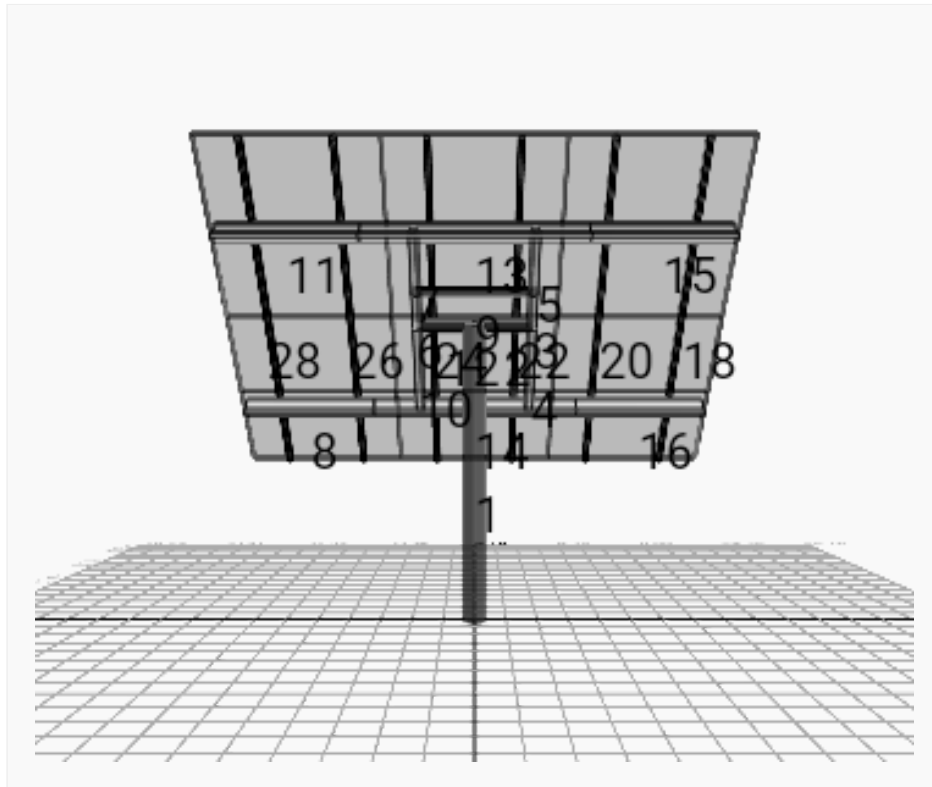
Number of Modules: 12

Unique ID: 1P-0-8TOP-HD-57-L-4Hx3W-8162

Number of Poles: 1

Dealer: _____

Date Sold: _____



Array Dimensions N/S	13.79 ft
Array Dimensions E/W	17.52 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

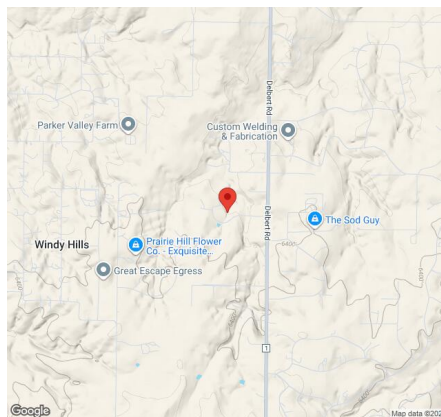
MT Solar Bill of Materials (1P-0-8TOP-HD-57-L-4Hx3W-8162)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-HD	H-Frame Assembly-HD	1
MTS-HD-Wing-57	57IN HD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	3

Rail Bill of Materials

Part	Qty
Rails (165in)	6
Rail Attachment	12
Module Mid Clamp	18
Module End Clamp	12
Ground Lug	3

Site Details:



Site Address: 11769 Sunset Dr, Parker, CO 80138, USA

Array Specification

Duty Classification:	HD
Module Width:	40.87 in
Module Length:	69.10in
Number of Rows:	4
Number of Columns:	3
Total Number of Modules:	12
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	15.56 ft
Total Frame Length:	17.00 ft
Module Info/Notes:	Longi
Array Dimensions N/S:	13.79 ft
Array Dimensions E/W:	17.52 ft
Rail Length:	165.48 in
Rail Spacing:	2.92 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	10.28 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft
Foundation Volume:	3.407 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	11769 Sunset Dr, Parker, CO 80138, USA
Wind Speed:	101 mph
Snow Load:	20 psf

Design Disclaimer

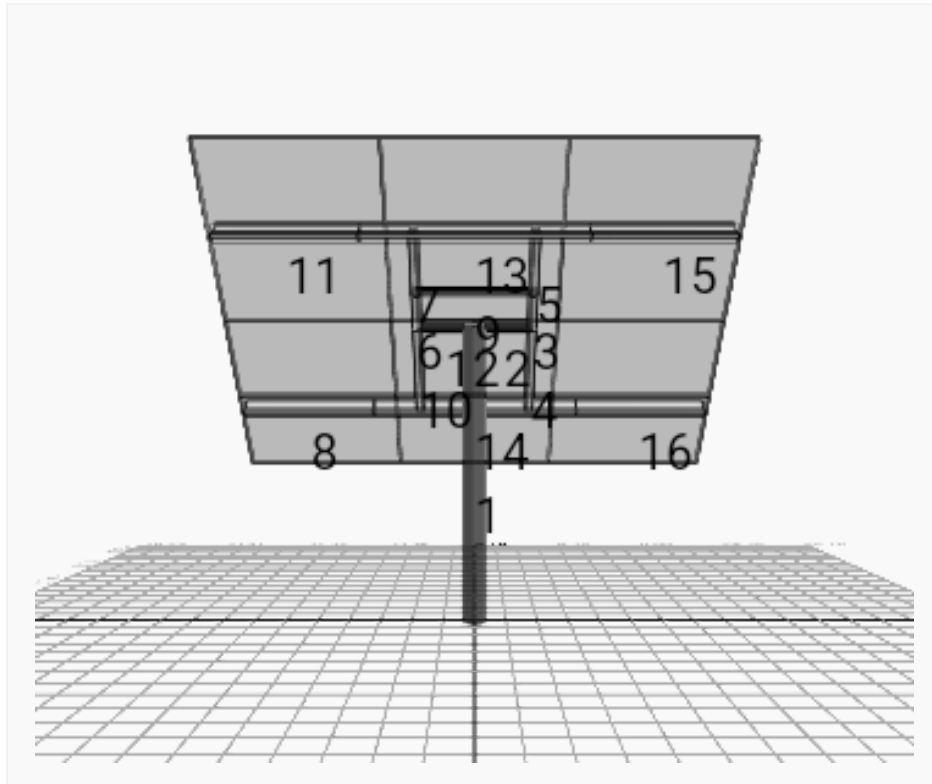
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

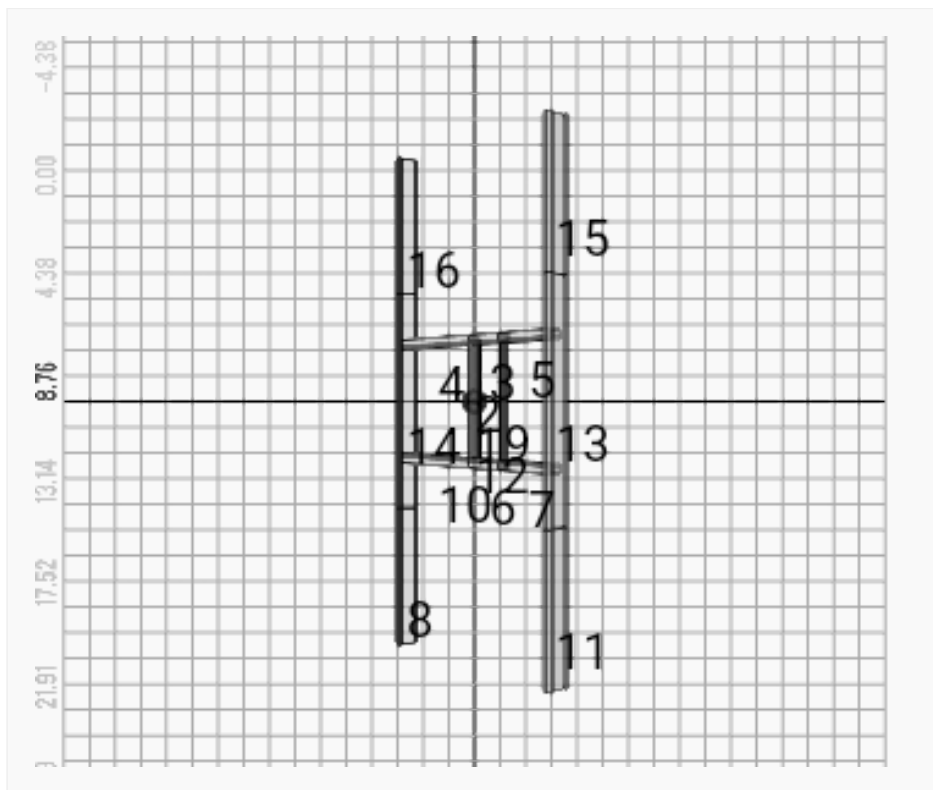
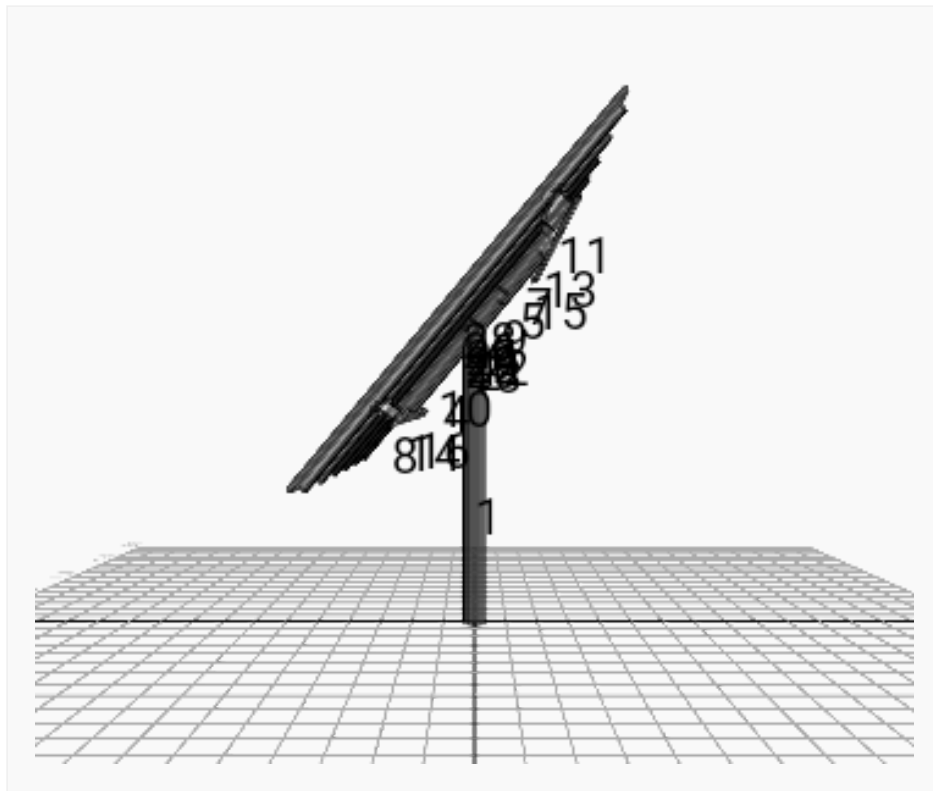
AutoDesigner Input

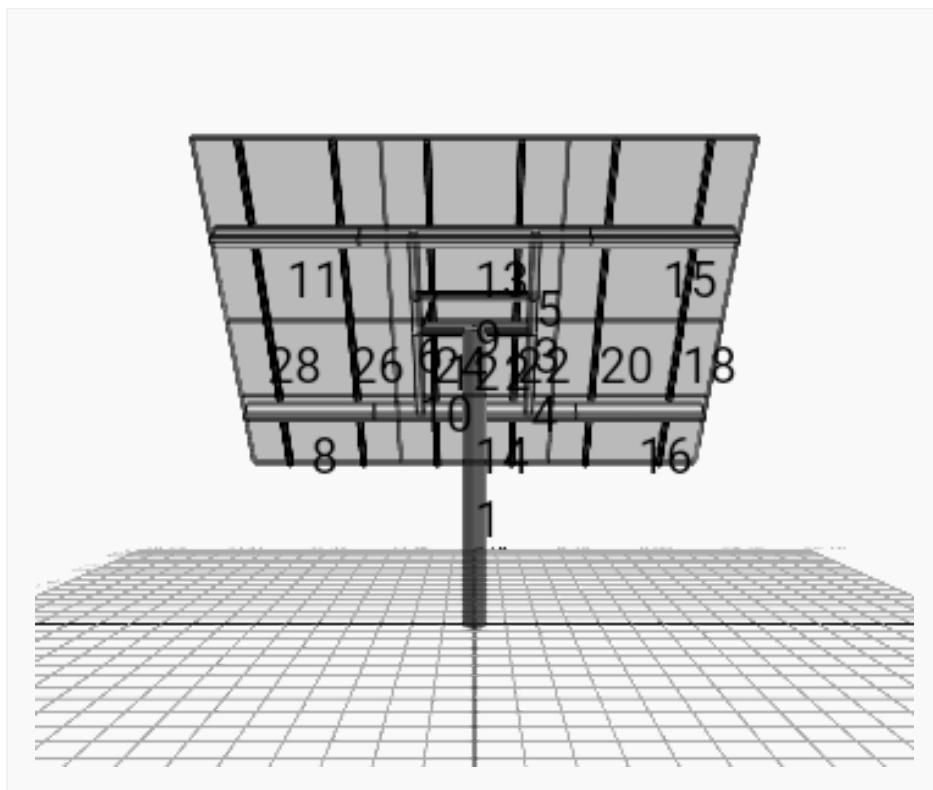
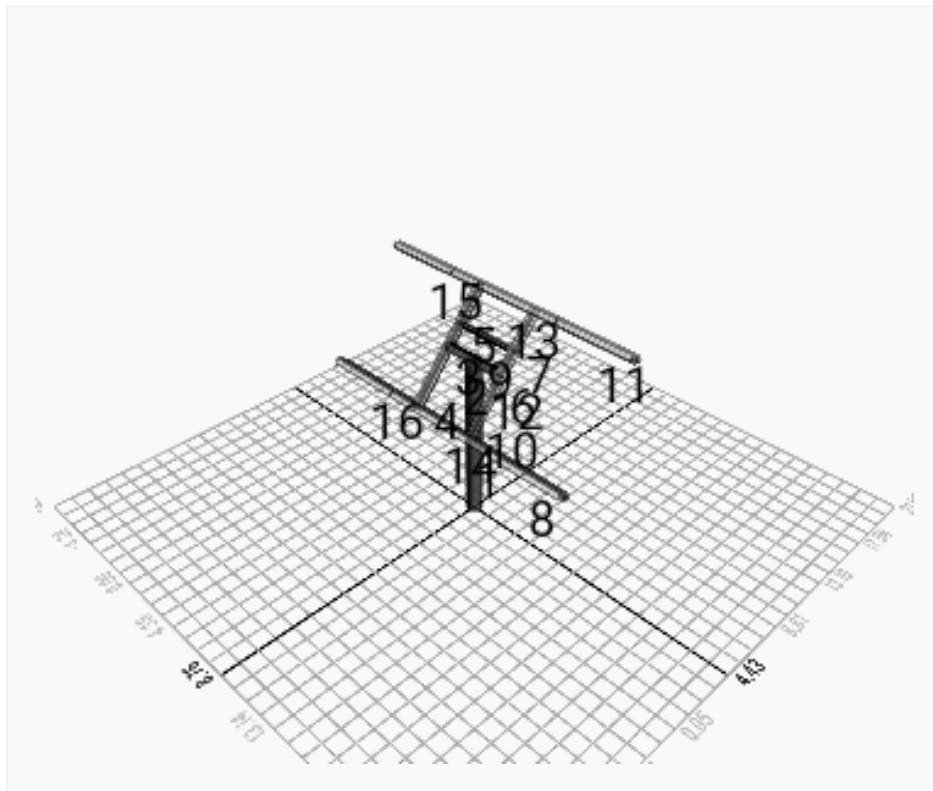
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Design Notes:

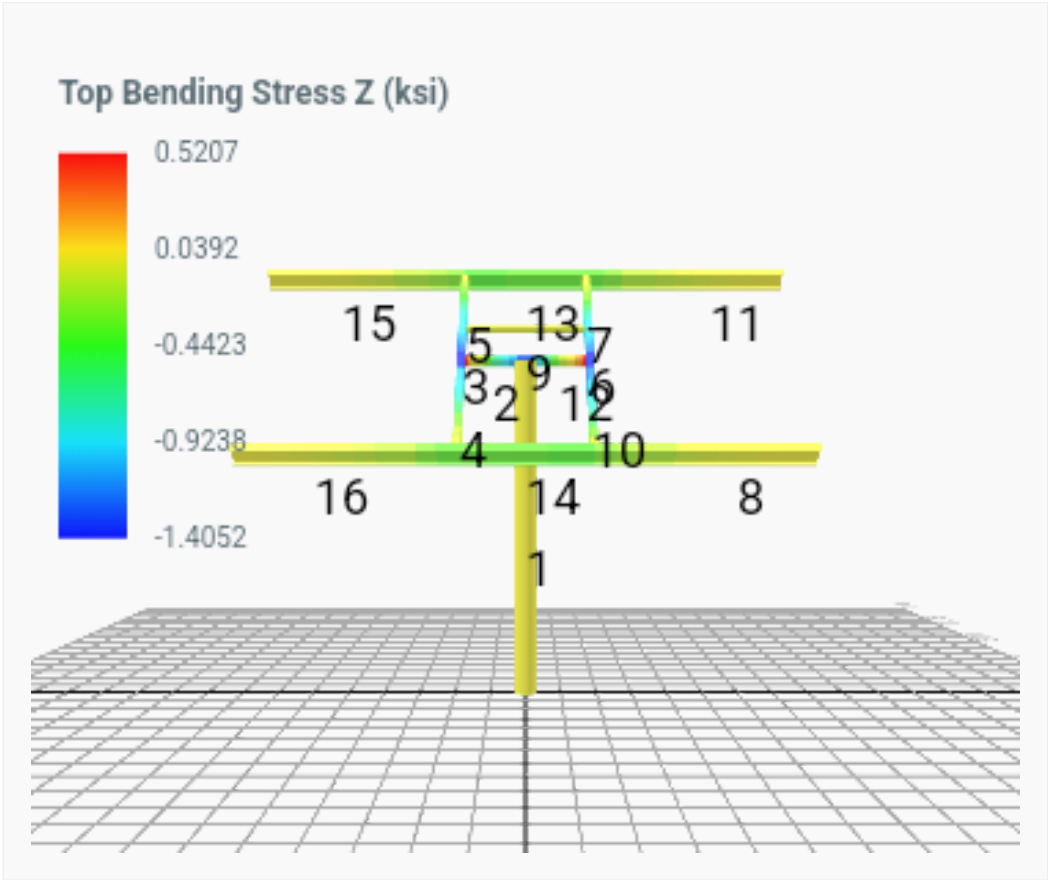
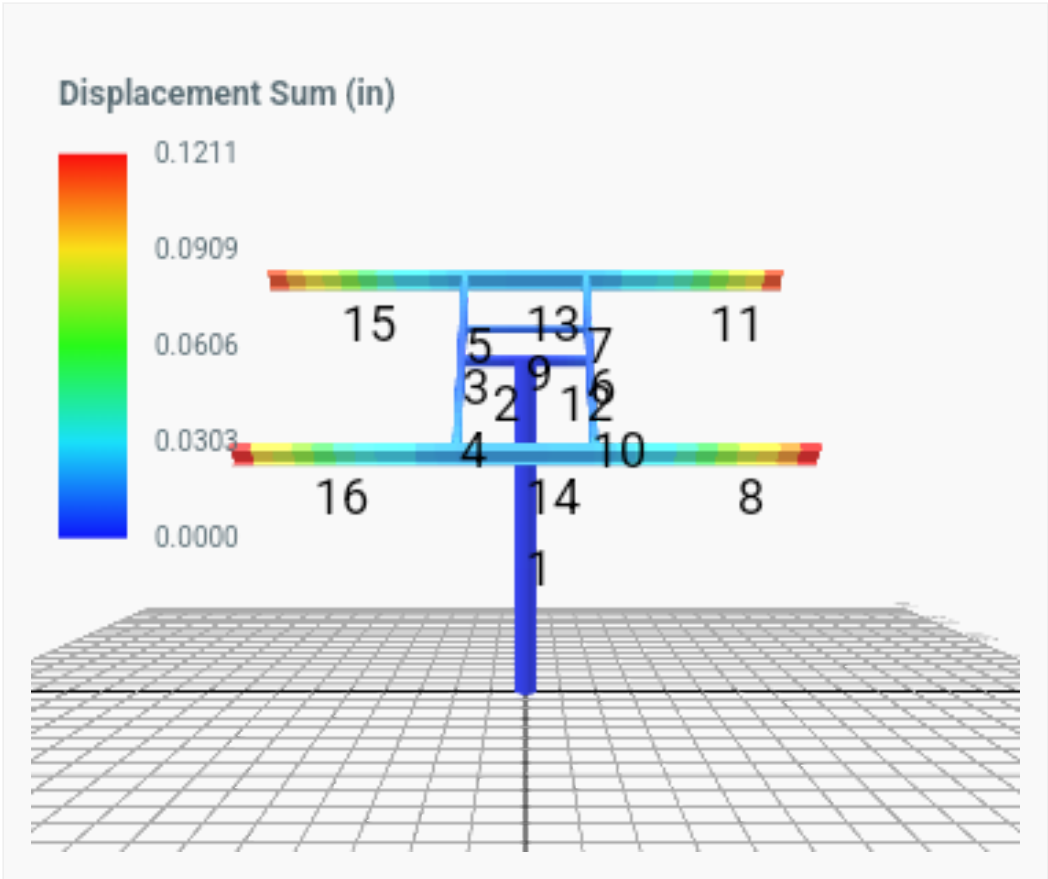
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only

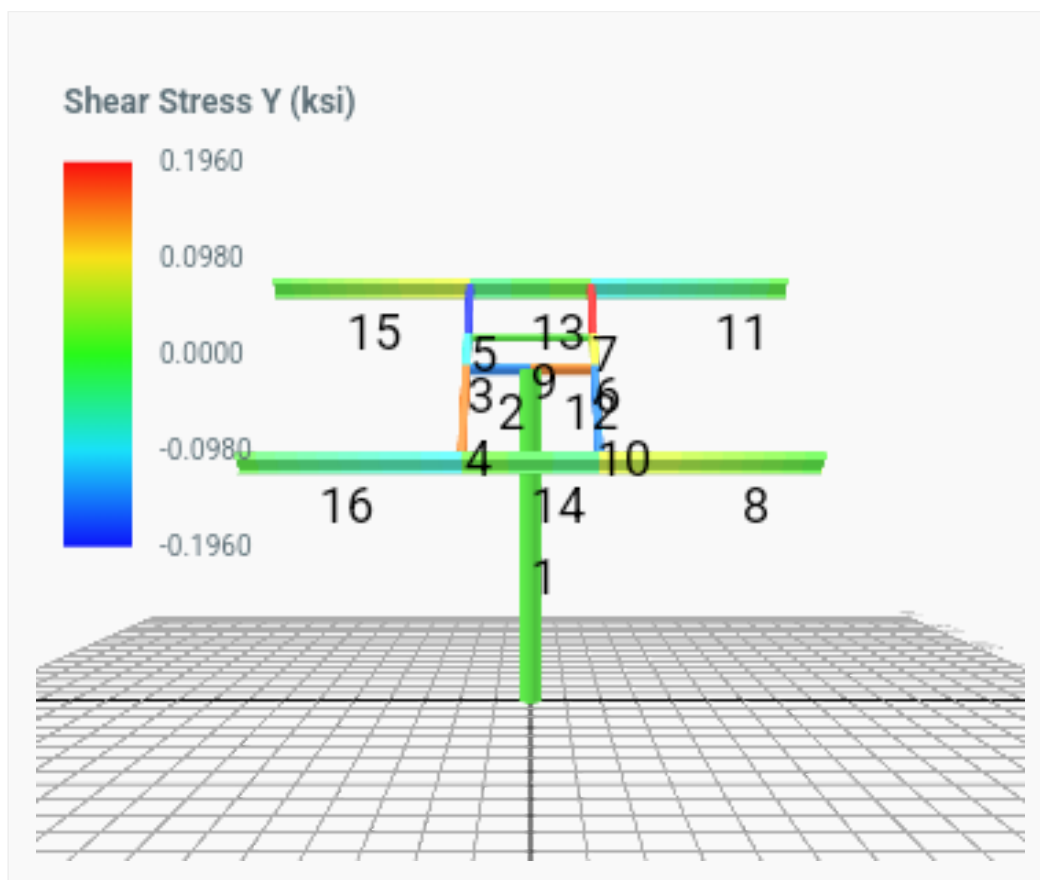
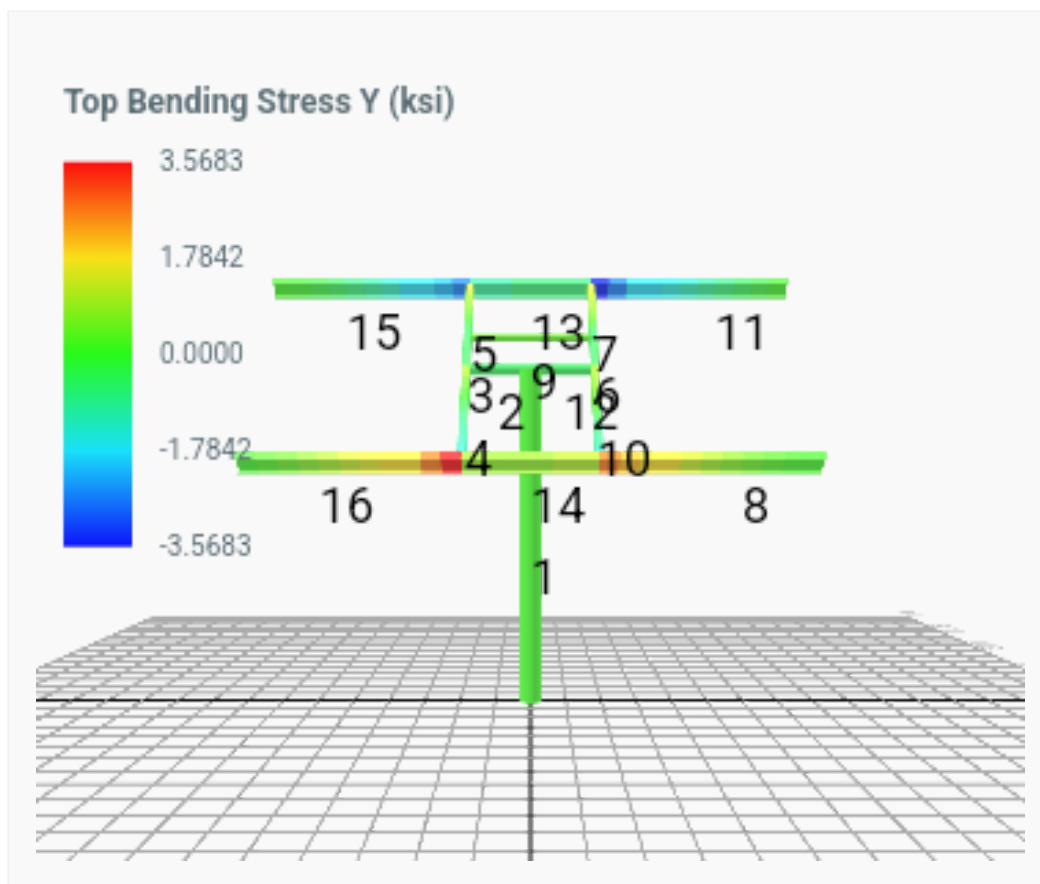


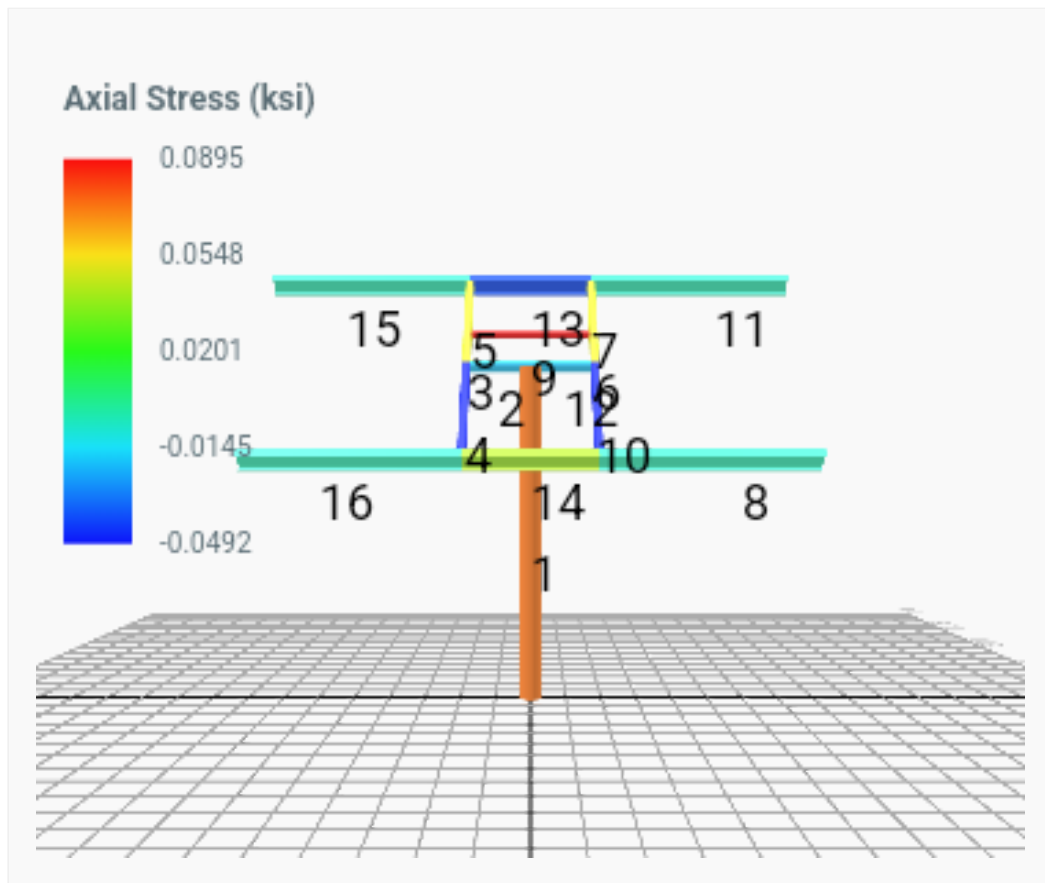




FEM Results (Envelope Worst Case for each member)







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	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 2. D + L	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 3. D + (S or Lr or R)	0.0000	2.6576	0.0000	0.0000	-0.0000	0.0208
ULS: 3. D + (S or Lr or R)	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.4919	0.0000	0.0000	-0.0000	0.0206
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 5b. D + 0.7E	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.4919	0.0000	0.0000	-0.0000	0.0206
ULS: 8. 0.6D + 0.7E	0.0000	1.1969	0.0000	0.0000	-0.0000	0.0121
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7873	3.4945	0.0000	0.0000	-0.0000	18.5089
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7873	0.4950	0.0000	0.0000	-0.0000	-18.2467
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3405	3.6167	0.0000	0.0000	-0.0000	13.8872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.4919	0.0000	0.0000	-0.0000	0.0206
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3405	1.3671	0.0000	0.0000	-0.0000	-13.6795
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.4919	0.0000	0.0000	-0.0000	0.0206
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3405	3.1196	0.0000	0.0000	-0.0000	13.8867
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3405	0.8700	0.0000	0.0000	-0.0000	-13.6799
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.9948	0.0000	0.0000	-0.0000	0.0202
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7873	2.6966	0.0000	0.0000	-0.0000	18.5008
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.1969	0.0000	0.0000	-0.0000	0.0121
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7873	-0.3029	0.0000	0.0000	-0.0000	-18.2548
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.1969	0.0000	0.0000	-0.0000	0.0121

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	5.2247
Shear X	-2.9789
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	31.0665

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	3.6167
Shear X	-1.7873
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	18.5089

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)					
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)
-	-	-	-	-	-	-	-	-

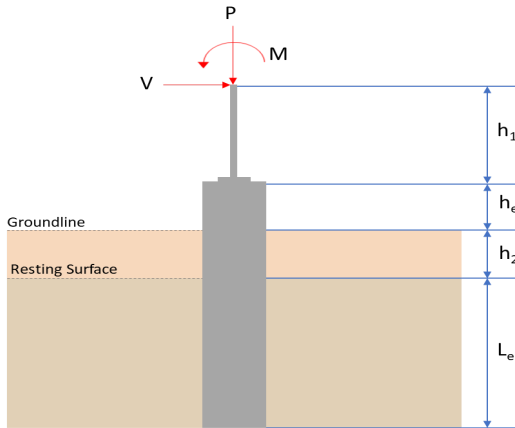
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62

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	0.024	0.373	0.000	0.026	0.000	0.385	#13	0.441	Not Required	Pass
2	0.002	0.180	0.138	0.041	0.025	0.319	#13	0.035	Not Required	Pass
3	0.005	0.317	0.026	0.032	0.002	0.339	#13	0.045	Not Required	Pass
4	0.005	0.316	0.093	0.032	0.019	0.362	#13	0.080	Not Required	Pass
5	0.005	0.196	0.096	0.032	0.024	0.215	#13	0.074	Not Required	Pass
6	0.005	0.317	0.026	0.032	0.002	0.339	#13	0.045	Not Required	Pass
7	0.005	0.196	0.096	0.032	0.024	0.215	#13	0.074	Not Required	Pass
8	0.000	0.053	0.118	0.018	0.007	0.156	#21	Not Required	Not Required	Pass
9	0.009	0.022	0.031	0.001	0.000	0.056	#13	0.204	Not Required	Pass
10	0.005	0.316	0.093	0.032	0.019	0.362	#13	0.080	Not Required	Pass
11	0.000	0.053	0.118	0.018	0.007	0.156	#21	Not Required	Not Required	Pass
12	0.002	0.180	0.138	0.041	0.025	0.319	#13	0.035	Not Required	Pass
13	0.004	0.145	0.221	0.025	0.010	0.321	#21	0.190	Not Required	Pass
14	0.005	0.147	0.221	0.025	0.010	0.321	#21	0.190	Not Required	Pass
15	0.000	0.053	0.118	0.018	0.007	0.156	#21	Not Required	Not Required	Pass
16	0.000	0.053	0.118	0.018	0.007	0.156	#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
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 in - Pile width D = 48 in - Pile depth L = 5.75 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>3.617</td><td>5.225</td></tr><tr><td>Vx (kip)</td><td>-1.787</td><td>-2.979</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>18.509</td><td>31.066</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)	3.617	5.225	Vx (kip)	-1.787	-2.979	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	18.509	31.066	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, $H_o = \frac{V_x}{1.57 \text{ } D}$ $H_o = \frac{(-1.787 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.28455 \text{ kip/ft}$	
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																										
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Vx (kip)	-1.787	-2.979																											
Vz (kip)	0.000	0.000																											
Mx (kipft)	0.000	0.000																											
Mz (kipft)	18.509	31.066																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(18.509 \text{ kipft}) + ((-1.787 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 2.9473 \text{ kipft/ft}$ 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: $L_{e,x} = 5.2645 \text{ ft}$ - 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, $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(5.2645 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.264 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.75 \text{ ft}$ Ratio - Embedded depth $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(5.264 \text{ ft})}{(5.75 \text{ ft})}$ $\text{Ratio} = 0.91548$	Status: PASS Ratio: 0.920
	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 $q = \frac{P_v}{A}$ $q = \frac{(3.617 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.22606 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.22606 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.11303$	Status: PASS Ratio: 0.110
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.28455 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9473 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9473 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.28455 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.9473 \text{ kipft/ft})) + (4 \times (-0.28455 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9628 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9473 \text{ kipft/ft})) + (3 \times (-0.28455 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (2.9473 \text{ kipft/ft})) + (2 \times (-0.28455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.19282 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9473 \text{ kipft/ft})) + ((-0.28455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.77279 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9628 \text{ ft})}{2}$$

$$p_a = 0.29721 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19282 \text{ kip/ft}^2)}{(0.29721 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.64879$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

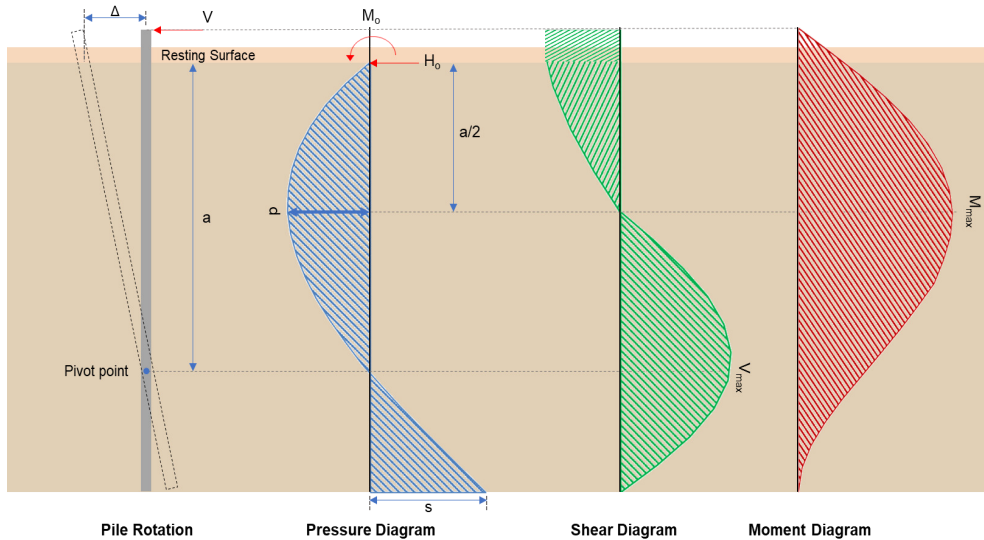
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.77279 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.650**

$$Ratio = 0.896$$

Status: **PASS**
Ratio: **0.900**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.979 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.47436 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.066 \text{ kipft}) + ((-2.979 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9468 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.9468 \text{ kipft/ft})}{(-0.47436 \text{ kip/ft})}$$

$$E = 10.428 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9468 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.47436 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.9468 \text{ kipft/ft})) + (4 \times (-0.47436 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9621 \text{ ft}$$

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

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

$$V_{max} = ((-0.47436 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.428 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9621 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.428 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9621 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 1.3411 \text{ kip}$ M_{max} - Max bending moment located at depth $a/2$, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.47436 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(10.428 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9621 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.428 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9621 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.428 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9621 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 20.115 \text{ kipft}$	
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, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(5.225 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.422 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.422 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ n_{rebar} - Required number of reinforcement, $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ A_{st} - Actual total reinforcement area, $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ Ratio - Capacity $\text{Ratio} = \frac{A_{min}}{A_{st}}$ $\text{Ratio} = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $\text{Ratio} = 0.96556$ 25.2.3 s_{rebar} - Minimum spacing of reinforcement, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$	Status: PASS Ratio: 0.970

	$s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$$s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), Min (D, b)]$$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min ((48 \text{ in}), (48 \text{ in}))]$$s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in	
22.4.2.2	Axial Compression Strength (ACI 318-19, LRFD) ϕP_N - Allowable axial compressive strength $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$\phi P_N = 2675.2 \text{ kip}$ Ratio - Capacity $Ratio = \frac{P}{\phi P_N}$$Ratio = \frac{(5.225 \text{ kip})}{(2675.2 \text{ kip})}$$Ratio = 0.0019531$	Status: PASS Ratio: 0.000
22.5.2.2	Shear Strength (ACI 318-19, LRFD) Parameters: $b_w = 48 \text{ in}$ - Effective width, d - Effective depth $d = 0.80 D$$d = 0.80 \times (48 \text{ in})$$d = 38.4 \text{ in}$	
22.5.5.1.3	λ_s - size effect modification factor $\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$$\lambda_s = 0.64282$	
22.5.5.1.1	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,max}$ - Max shear strength of concrete $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$V_{c,max} = 296.21 \text{ kip}$	
22.5.5.1.1(a)	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.225 \text{ kip} \rightarrow 5225 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(5225 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.18 \text{ kip}$$

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)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.18 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.18 \text{ kip}$$

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

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.18 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.55 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.3411 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.3411 \text{ kip})}{(110.55 \text{ kip})}$$

$$Ratio = 0.066406$$

Status: **PASS**
Ratio: **0.070**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$

$$\phi M_n = 249.6 \text{ kipft}$$

Considering x-direction:

$M_{max} = 20.115 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(20.115 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.080587$$

Status: **PASS**
Ratio: **0.080**