

Your Project Calculations



Project Name: Lohman-CU-B

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Lohman-CU-B&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=tXSEnUNSc6hrvCmnfTJHLp17IfiVW2fOOjn4NkAeW00XHt6ZvwiYPlxsUqTCnbIF

Array Specification

Product:	Beam
Unique ID:	1P-0-10TOP-XD-84-L-4Hx4W-BGE9
Duty Classification:	XD
Module Width:	40.80 in
Module Length:	69.40in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	15.48 ft
Total Frame Length:	21.50 ft
Frame Weight:	1205 lbs
Array Dimensions N/S:	13.77 ft
Array Dimensions E/W:	23.47 ft
Rail Length:	165.20 in
Rail Spacing:	2.93 ft
Rail Check:	PASS (50% utilized)

Support Specifications

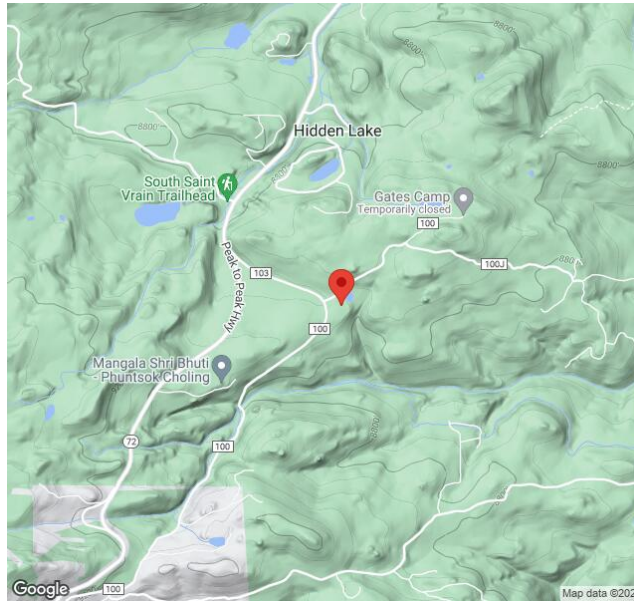
Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	10.27 ft
Number of Poles:	1
Pole Spacing:	0

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.25 ft
Foundation Volume:	4.889 y ³
Foundation Result:	PASSED
Mount Twist:	0.000008 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2000 Co Rd 100, Ward, CO 80481, USA
Wind Speed:	175 mph
Snow Load:	55 psf
Design Uplift Pressure:	0.046469 ksf
Design Downforce Pressure:	-0.046469 ksf
Design Snow Pressure:	0.012096 ksf



Design Disclaimer

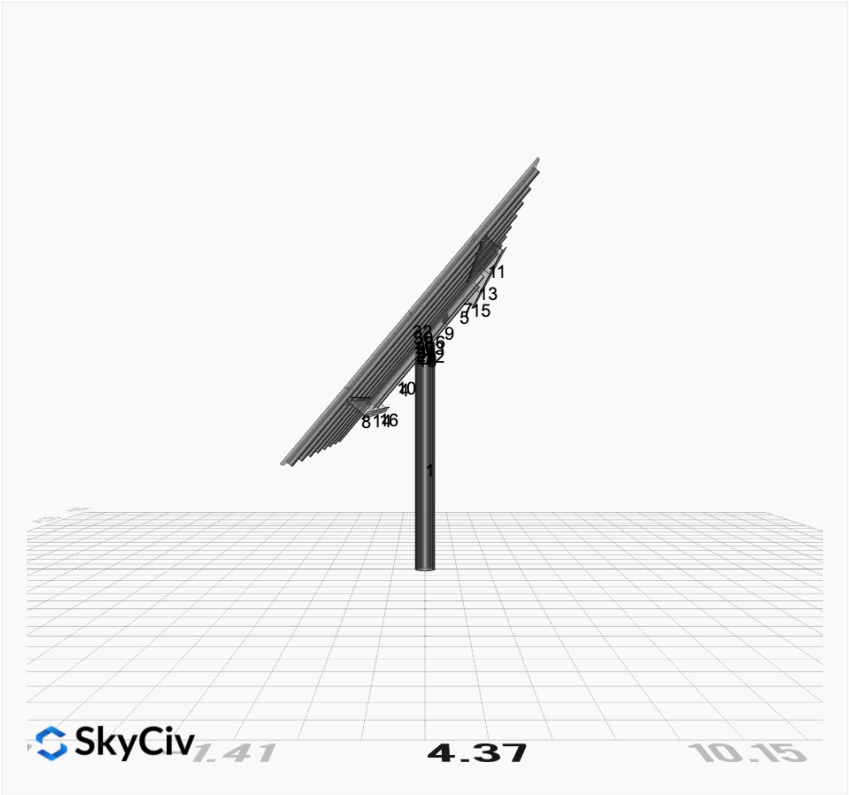
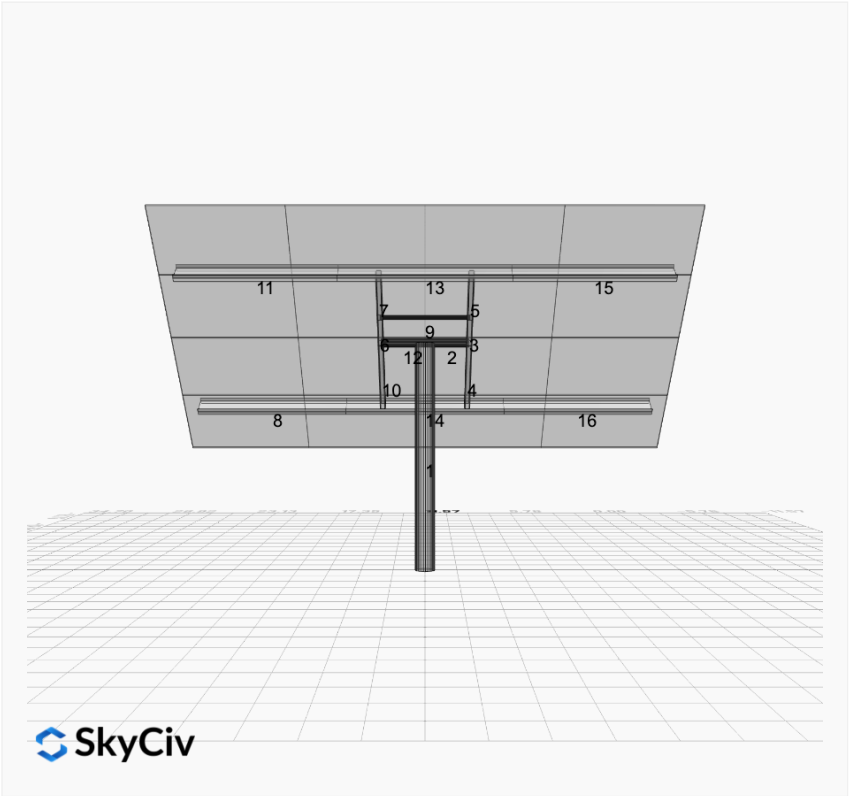
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

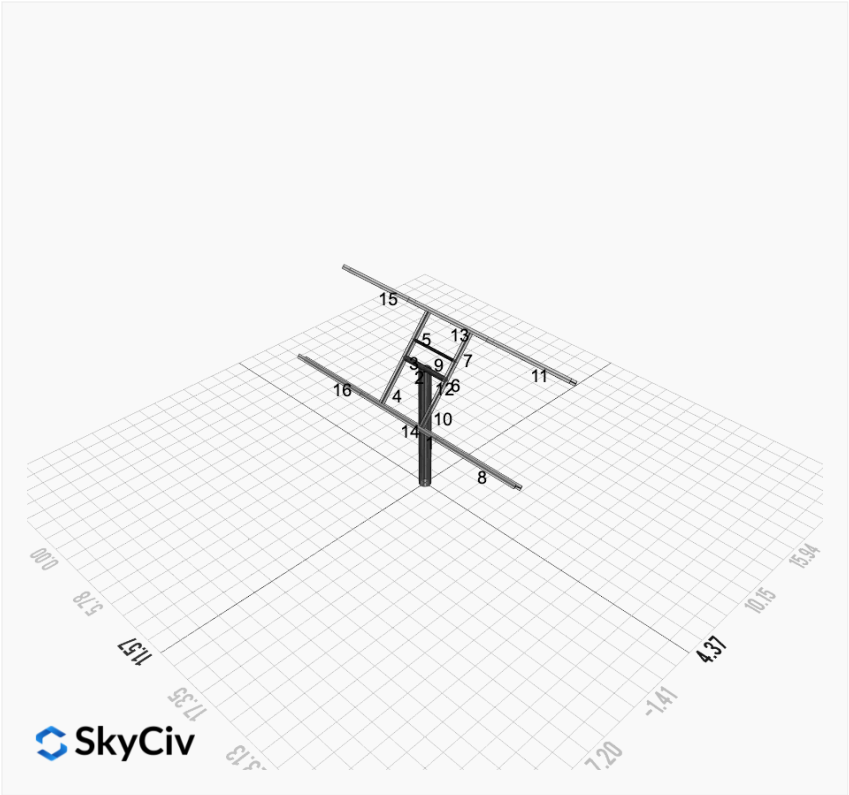
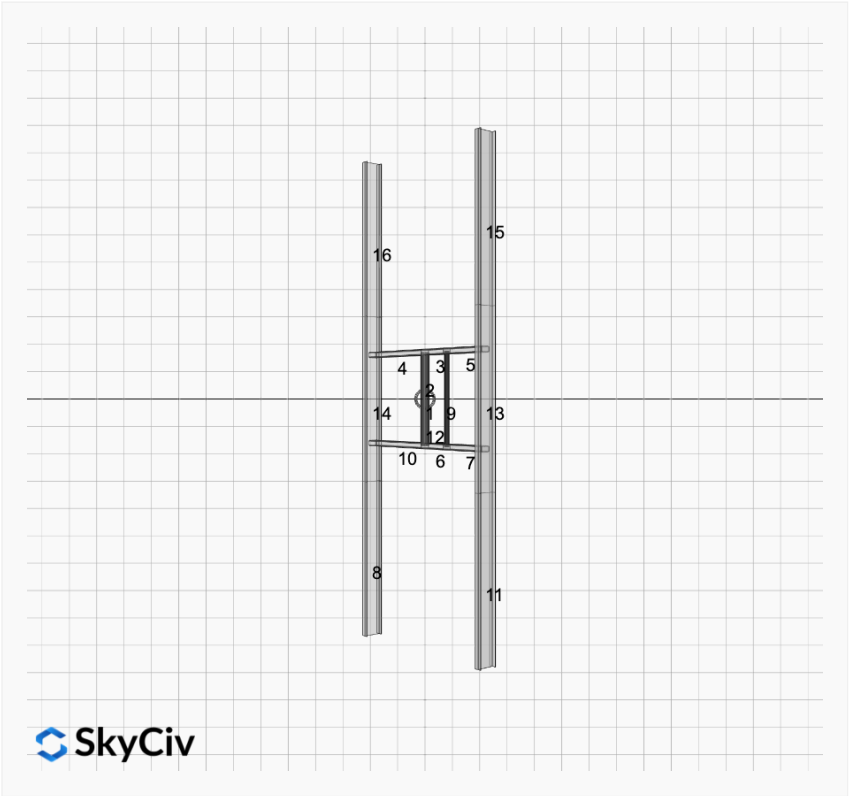
AutoDesigner Input

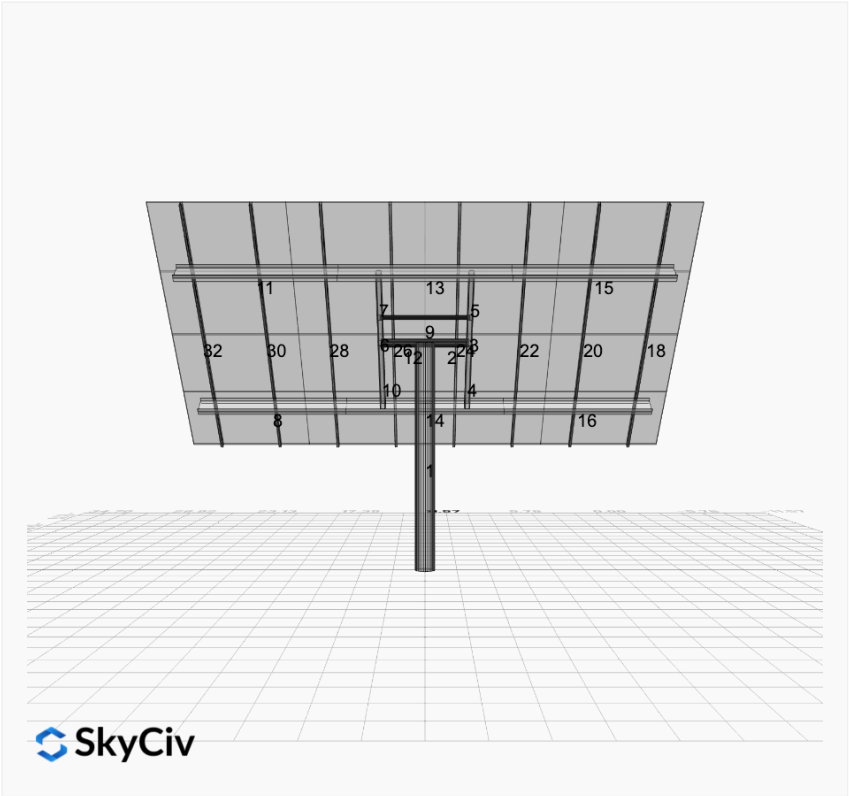
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Design Notes:

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

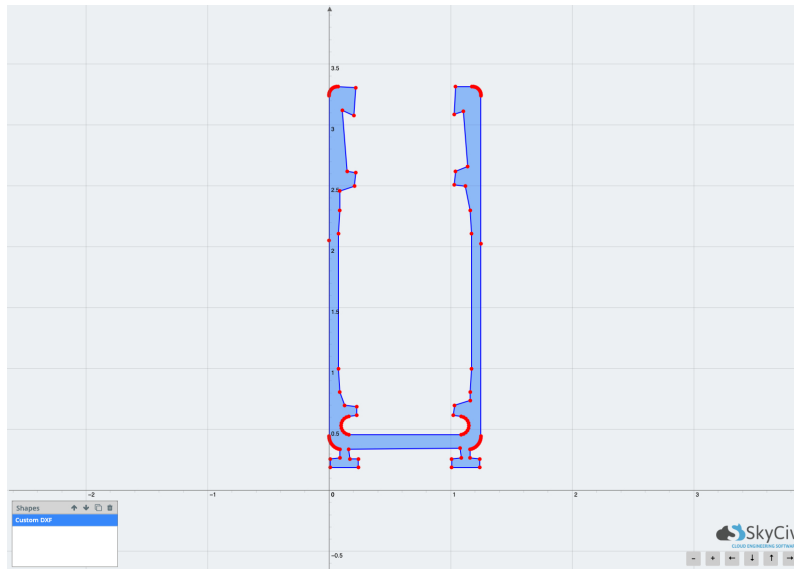






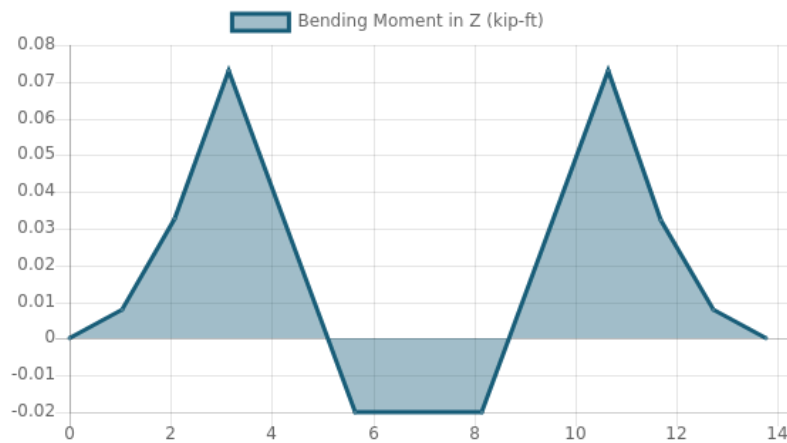
Rail Design Check

Rail Length: 13.766666666666666 ft
Additional Restraints Required: None
Tributary Width: 2.9333333333333336 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0228 kip/ft
Snow (Y): -0.0272 kip/ft
Wind uplift Case A: 0.1363 kip/ft
Wind downforce Case A: 0.1363 kip/ft
Dead (Panel load) (X): 0.0089 kip/ft
Dead (Panel load) (Y): -0.0107 kip/ft

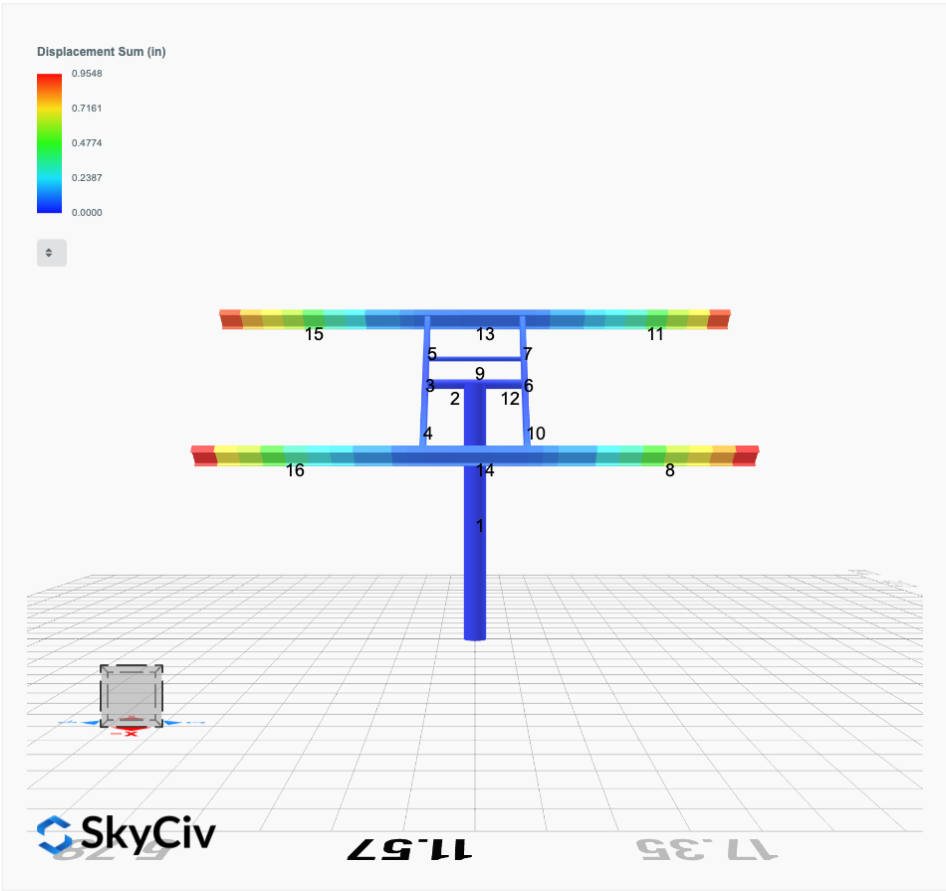


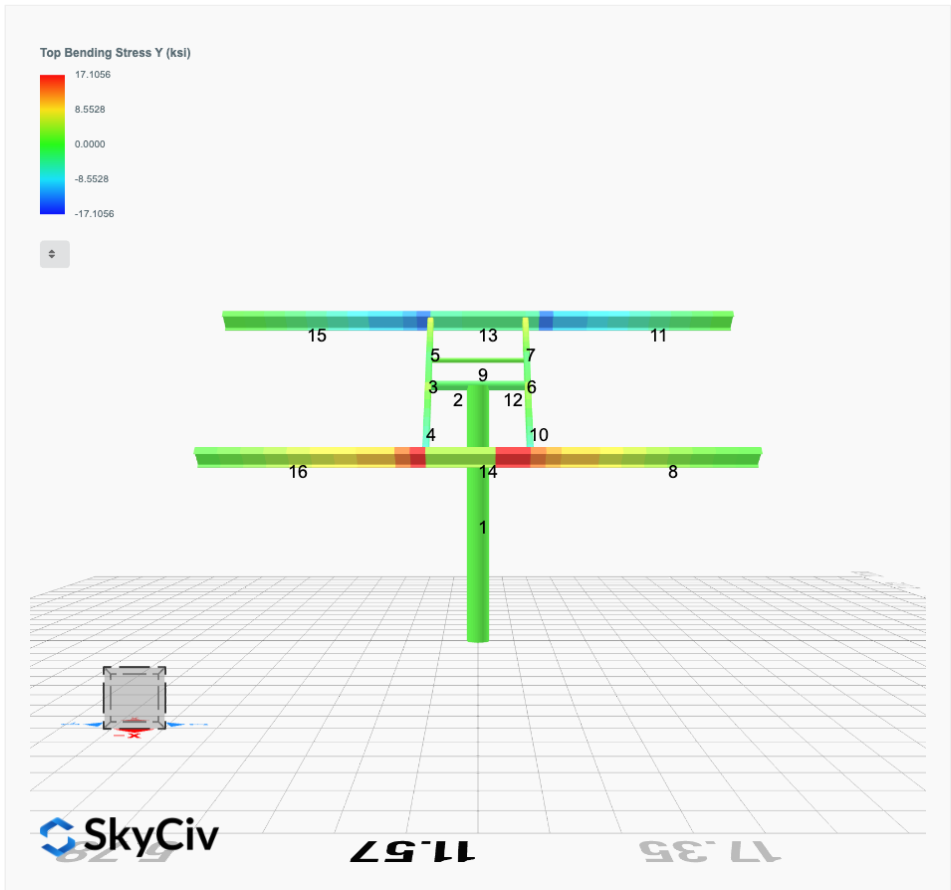
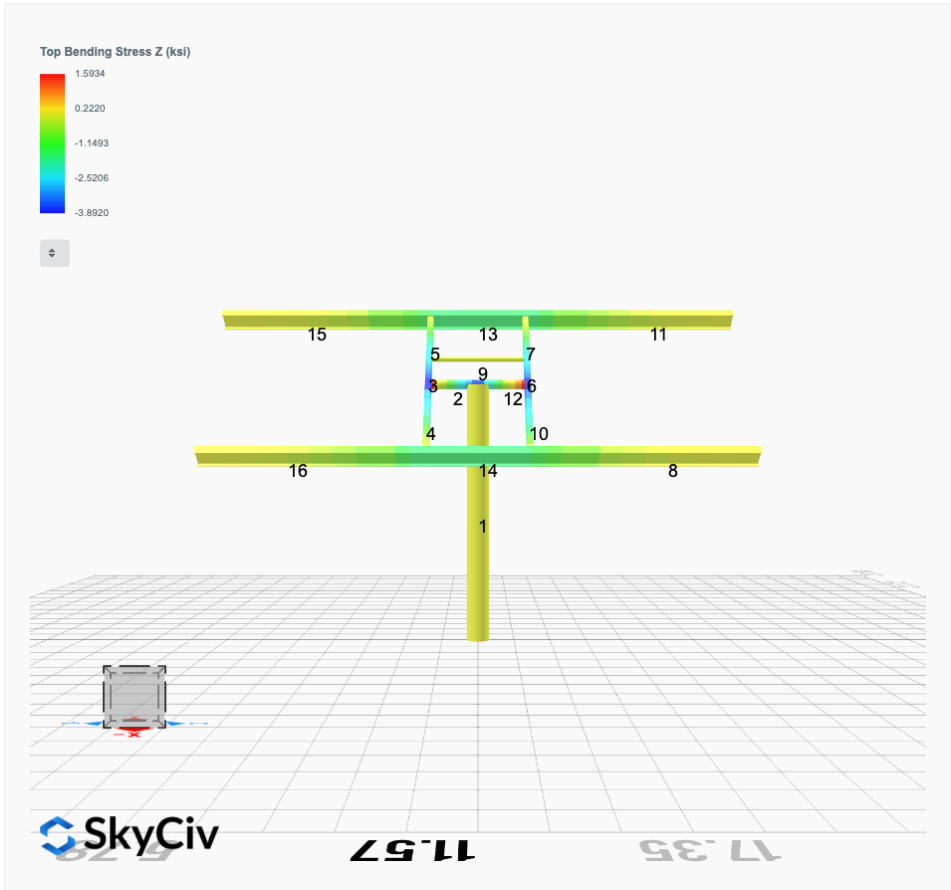
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	17.31848032	0.502	PASS
Material Yield	34.5	17.31848032	0.502	PASS
Material Strength	37	17.31848032	0.468	PASS

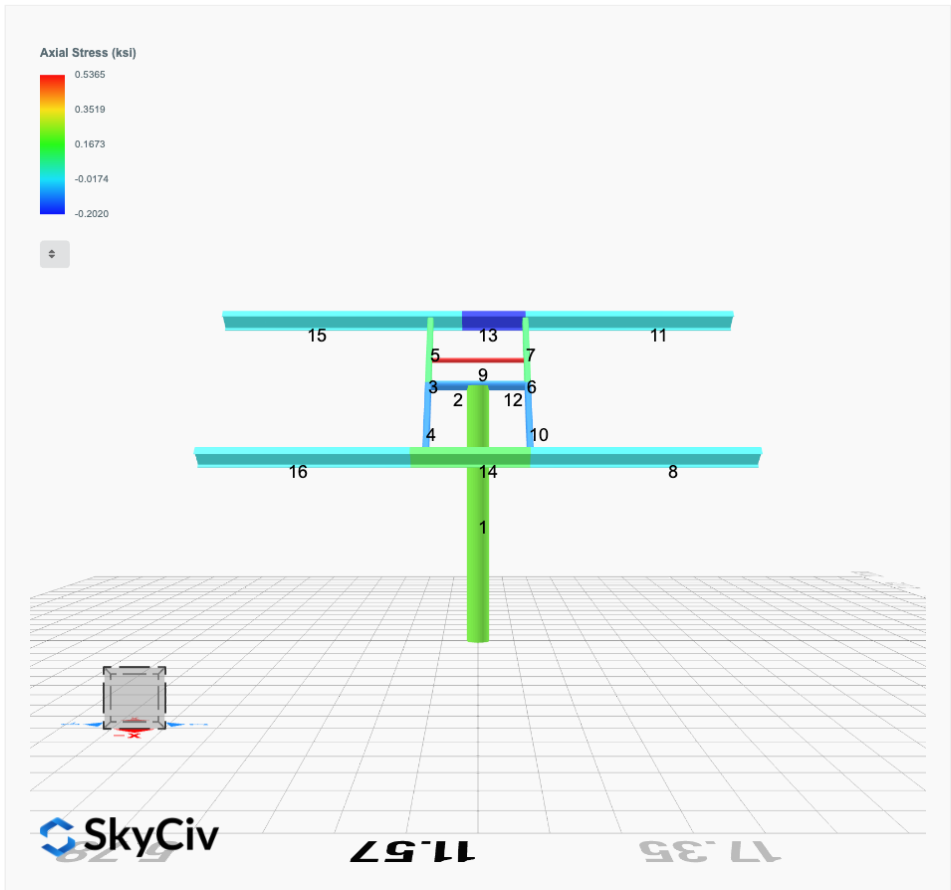
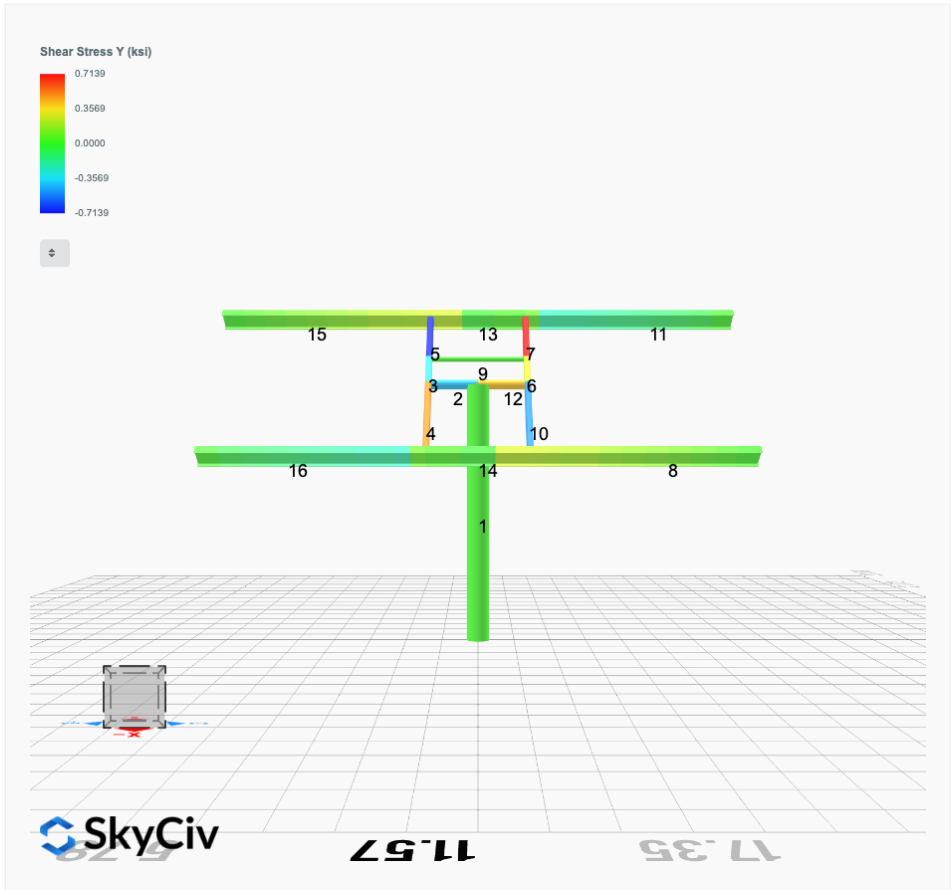
Member 1, ULS: 1.1.4D



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	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 2. D + L	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 3. D + (S or Lr or R)	0.0000	5.0384	0.0000	0.0000	-0.0000	0.0288
ULS: 3. D + (S or Lr or R)	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.4631	0.0000	0.0000	-0.0000	0.0274
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 5b. D + 0.7E	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.4631	0.0000	0.0000	-0.0000	0.0274
ULS: 8. 0.6D + 0.7E	0.0000	1.6423	-0.0000	0.0000	-0.0000	0.0141
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.9000	8.5269	0.0000	0.0000	-0.0000	71.6595
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.9000	-3.0527	0.0000	0.0000	-0.0000	-70.1257
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.1750	8.8055	0.0000	0.0000	-0.0000	53.7545
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.4631	0.0000	0.0000	-0.0000	0.0274
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.1750	0.1208	0.0000	0.0000	-0.0000	-52.5844
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.4631	0.0000	0.0000	-0.0000	0.0274
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.1750	7.0795	0.0000	0.0000	-0.0000	53.7505
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.1750	-1.6052	0.0000	0.0000	-0.0000	-52.5884
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.7371	-0.0000	0.0000	-0.0000	0.0234
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.9000	7.4321	0.0000	0.0000	-0.0000	71.6501
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.6423	-0.0000	0.0000	-0.0000	0.0141
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.9000	-4.1475	0.0000	0.0000	-0.0000	-70.1351
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.6423	-0.0000	0.0000	-0.0000	0.0141

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	14.0849
Shear X	-11.5000
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	120.1103

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	8.8055
Shear X	-6.9000
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	71.6595

Project Details

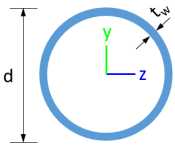
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 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial



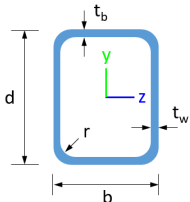
Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

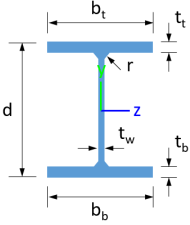
Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

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					
11	10in Pipe Sch 40	10.75	0.36					

								
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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		

								
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ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38

17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L S T	L S C	L D	
1	11	21.57	21.57	10.27	-	300	200	1	
2	6	1.30	1.30	2.00	-	300	200	1	
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1	
4	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1	
5	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1	
6	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1	
7	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68	300	200	1	
8	20	14.70	14.70	7.00	2.33,2.33	300	200	1	
9	3	2.60	2.60	4.00	-	300	200	1	
10	17	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69	300	200	1	
11	20	14.70	14.70	7.00	2.33,2.33	300	200	1	
12	6	1.30	1.30	2.00	-	300	200	1	
13	20	4.88	4.00	7.50	1.02,1.02	300	200	1	
14	20	4.88	4.00	7.50	1.02,1.02	300	200	1	
15	20	14.70	14.70	7.00	2.33,2.33	300	200	1	
16	20	14.70	14.70	7.00	2.33,2.33	300	200	1	

Member Design Capacity

Member ID	$\Phi_c P_n$ (kip)	$\Phi_t P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	535.87	372.73	147.68	147.68	160.76	160.76
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	15.83	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95

11	159.30	15.83	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	31.17	6.46	56.26	44.91
14	159.30	116.35	31.17	6.46	56.26	44.91
15	159.30	15.83	46.90	6.46	56.26	44.91
16	159.30	15.83	46.90	6.46	56.26	44.91

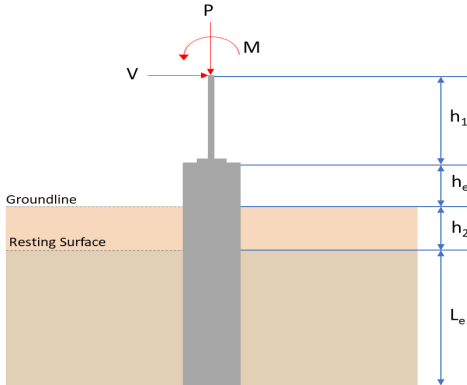
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.038	0.813	0.000	0.072	0.000	0.832	#13	0.352	Not Required	Pass
2	0.007	0.421	0.405	0.090	0.076	0.829	#13	0.036	Not Required	Pass
3	0.008	0.813	0.063	0.081	0.015	0.828	#13	0.046	Not Required	Pass
4	0.008	0.810	0.238	0.081	0.048	0.891	#13	0.082	Not Required	Pass
5	0.008	0.505	0.247	0.080	0.066	0.546	#13	0.076	Not Required	Pass
6	0.008	0.813	0.063	0.081	0.015	0.828	#13	0.046	Not Required	Pass
7	0.008	0.505	0.247	0.080	0.066	0.546	#13	0.076	Not Required	Pass
8	0.000	0.211	0.415	0.050	0.017	0.554	#21	Not Required	Not Required	Pass
9	0.035	0.059	0.081	0.001	0.000	0.150	#13	0.206	Not Required	Pass
10	0.008	0.810	0.238	0.081	0.048	0.891	#13	0.082	Not Required	Pass
11	0.000	0.211	0.415	0.050	0.017	0.554	#21	Not Required	Not Required	Pass
12	0.007	0.421	0.405	0.090	0.076	0.829	#13	0.036	Not Required	Pass
13	0.012	0.513	0.649	0.063	0.021	0.976	#21	0.204	Not Required	Warn
14	0.012	0.518	0.649	0.063	0.021	0.976	#21	0.204	Not Required	Warn
15	0.000	0.211	0.415	0.050	0.017	0.554	#21	Not Required	Not Required	Pass
16	0.000	0.211	0.415	0.050	0.017	0.554	#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

no 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 = 8.25 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>8.805</td><td>14.085</td></tr><tr><td>Vx (kip)</td><td>-6.900</td><td>-11.500</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>71.660</td><td>120.110</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)	8.805	14.085	Vx (kip)	-6.900	-11.500	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	0.000	Mz (kipft)	71.660	120.110	
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	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ Considering x-direction: H_o - Lateral force per length of pile, $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-6.9 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -1.0987 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(71.66 \text{ kipft}) + ((-6.9 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 11.411 \text{ kipft/ft}$ Required depth of embedment in earth: $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ Solving the cubic equation: $L_{e,x} = 7.4853 \text{ ft}$ - Required depth in x-direction, Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction, Minimum embedded depth required: $L_{e,req}$ - Depth of pile required, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(7.4853 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 7.485 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 8.25 \text{ ft}$ <i>Ratio</i> - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(7.485 \text{ ft})}{(8.25 \text{ ft})}$ $Ratio = 0.90727$	Status: PASS Ratio: 0.910
	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 $q = \frac{P_v}{A}$ $q = \frac{(8.805 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.55031 \text{ kip/ft}^2$ Check bearing capacity ratio: <i>Ratio</i> - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.55031 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.27516$	Status: PASS Ratio: 0.280
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio, $L/D = \frac{L}{D}$ $L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.0987$ kip/ft - Lateral force per length of pile,

$M_o = 11.411$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.411 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.0987 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (11.411 \text{ kipft/ft})) + (4 \times (-1.0987 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.738 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (11.411 \text{ kipft/ft})) + (3 \times (-1.0987 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^3 [(3 \times (11.411 \text{ kipft/ft})) + (2 \times (-1.0987 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.23293 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (11.411 \text{ kipft/ft})) + ((-1.0987 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.2128 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.738 \text{ ft})}{2}$$

$$p_a = 0.43035 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23293 \text{ kip/ft}^2)}{(0.43035 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.54125$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

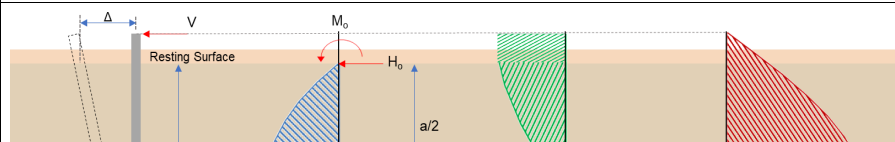
$$\text{Ratio} = \frac{s}{p_s}$$

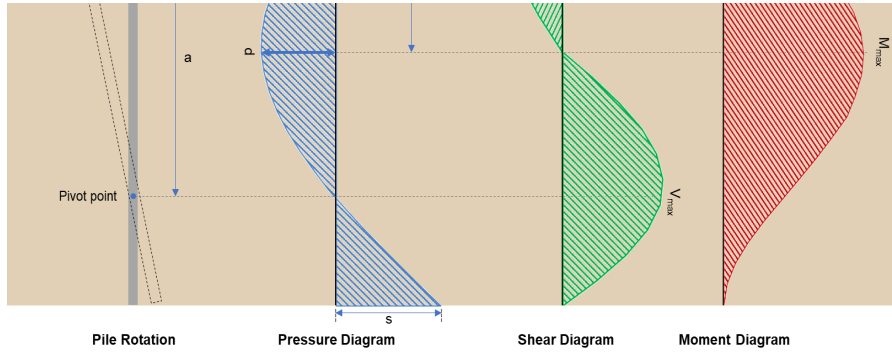
$$\text{Ratio} = \frac{(1.2128 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98$$

Status: **PASS**
Ratio: **0.540**

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-11.5 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.8312 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(120.11 \text{ kipft}) + ((-11.5 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 19.126 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(19.126 \text{ kipft/ft})}{(-1.8312 \text{ kip/ft})}$$

$$E = 10.444 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (19.126 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-1.8312 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (19.126 \text{ kipft/ft})) + (4 \times (-1.8312 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.7372 \text{ ft}$$

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

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

$$V_{max} = ((-1.8312 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.444 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.7372 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.444 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.7372 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.24 \text{ kip}$$

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

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

$$M_{max} = ((-1.8312 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(10.444 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.7372 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.444 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.7372 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.444 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.7372 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

		$M_{max} = 82.151 \text{ kipft}$	
		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, $\alpha = 0.8$ - Alpha factor for axial strength, $A_g = 2304 \text{ in}^2$ - Gross area of concrete, Longitudinal reinforcement: Required reinforcement due to axial load, $A_{st,required}$ $A_{st,required}$ $A_{st,required} = \text{Min} \left[\frac{\frac{P}{\phi \alpha} - (0.85 f'_{ck} A_g)}{f_{yk} - (0.85 f'_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = \text{Min} \left[\frac{\frac{(14.085 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -84.128 \text{ in}^2$ A_{min} - Governing minimum reinforcement area, $A_{min} = \text{Max} [A_{st,required}, (0.0018 A_g)]$ $A_{min} = \text{Max} [(-84.128 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ 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, $s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max} [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ Ties: 25.7.2.2 Since longitudinal reinforcement is $\leq \text{No. } 10\phi$: Use #3(0.375 in) 25.7.2.1 s_{ties} - Maximum spacing of ties, $s_{ties} = \text{Min} [(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ Summary: Main reinforcement: 14 - #5 (0.625 in)	Status: PASS Ratio: 0.970

	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 $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(14.085 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0052651$	Status: PASS Ratio: 0.010
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 $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$	
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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.085 \text{ kip} \rightarrow 14085 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14085 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 120.36 \text{ kip}$	
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 $V_c = \text{Min} [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min} [(296.21 \text{ kip}), (120.36 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 120.36 \text{ kip}$	

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) $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ V_s - Governing shear strength of steel $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ 22.5.1.1 ϕV_n - Allowable shear strength $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.36 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.32 \text{ kip}$ Considering x-direction: $V_{max} = 21.24 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(21.24 \text{ kip})}{(111.32 \text{ kip})}$ $Ratio = 0.1908$	Status: PASS Ratio: 0.190
14.5.2.1b	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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kip ft}$ $\phi M_{n,2}$	

	$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ 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: $M_{max} = 82.151 \text{ kipft}$ - Maximum moment in the x-direction, <i>Ratio</i> - Capacity $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(82.151 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.32913$	
		Status: PASS Ratio: 0.330