

Project Details



Project Name: Scofield 8IN 12TOP Struts - V1Jb

Location: Scofield, UT 84526, USA

Unique ID: 1P-0-8TOP-SD-84-L-4Hx4W-STRUTS-22II

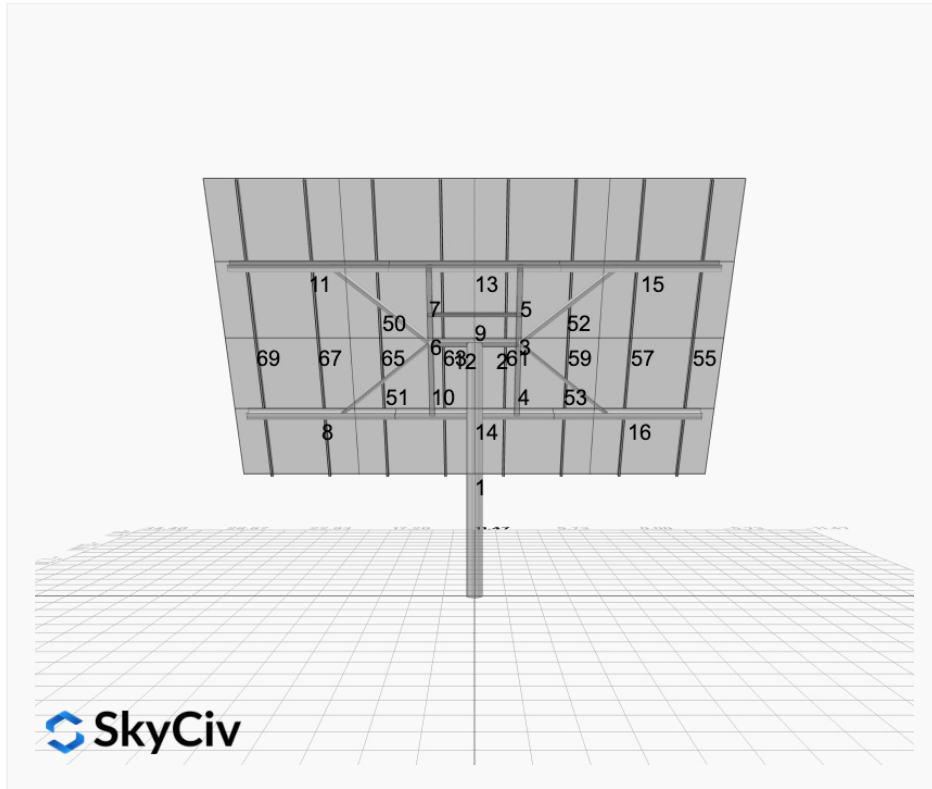
Dealer: _____

Date: Tue Sep 23 2025

Number of Modules: 16

Number of Poles: 1

Date Sold: _____



Array Dimensions N/S	15.05 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	60
Front Edge Clearance	5 ft

MT Solar Bill of Materials (1P-0-8TOP-SD-84-L-4Hx4W-STRUTS-22II)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	1
MTS-HF-SD	H-Frame Assembly-SD	1
MTS-SD-Wing-84	84IN SD Wing	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	4

Rail Bill of Materials

Part	Qty
Rails (181in)	8
Rail Attachment	16
Module Mid Clamp	24
Module End Clamp	16
Ground Lug	4

Site Details:



Site Address: Scofield, UT 84526, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	67.80in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Winter Tilt Angle:	60
Front Edge Clearance:	5
Total Array Height at Tilt:	18.03 ft
Total Frame Length:	21.50 ft
Module Info/Notes:	phono 440
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	22.93 ft
Rail Length:	180.60 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.52 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: 6.75 ft
Foundation Volume:	4.000 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Scofield, UT 84526, USA
Wind Speed:	97 mph
Snow Load:	160 psf

Design Disclaimer

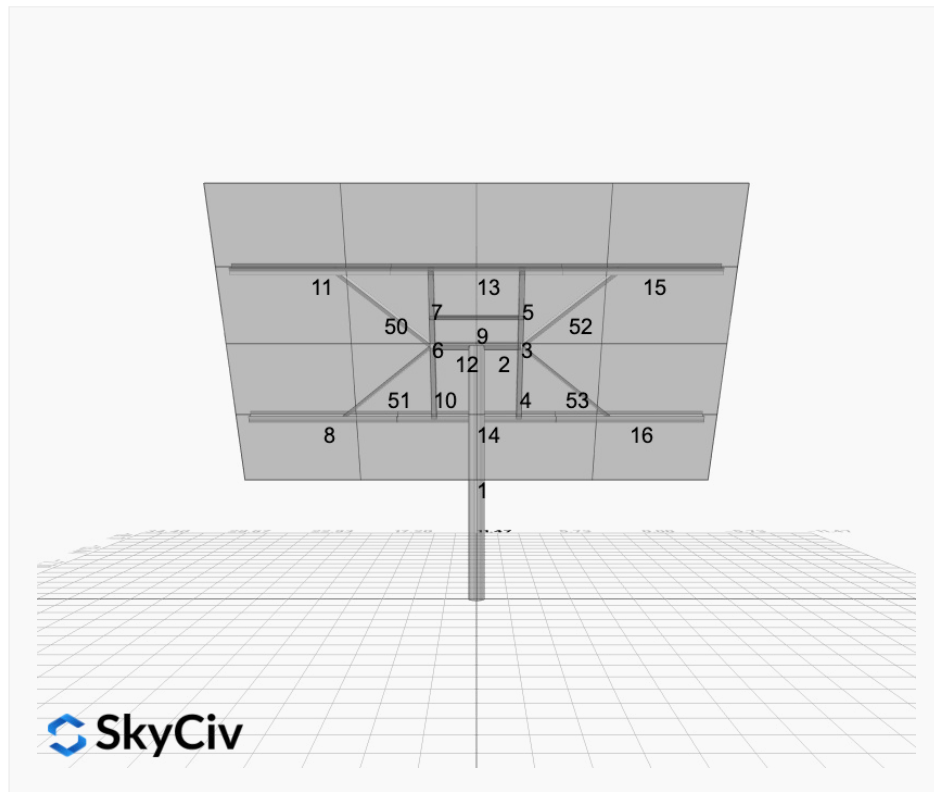
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

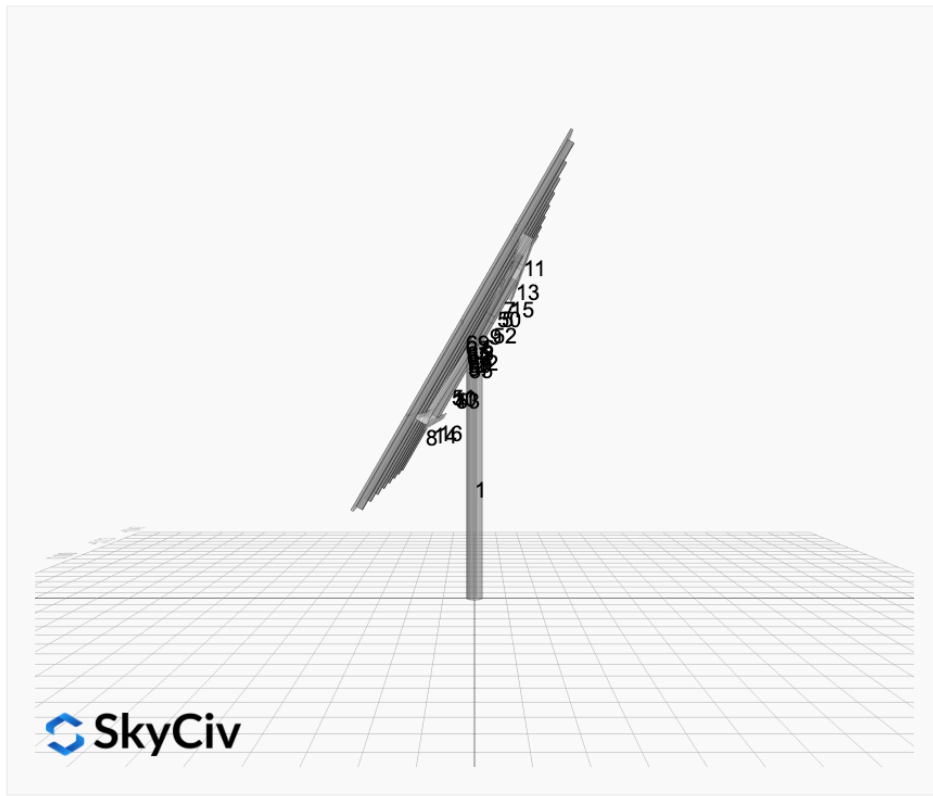
AutoDesigner Input

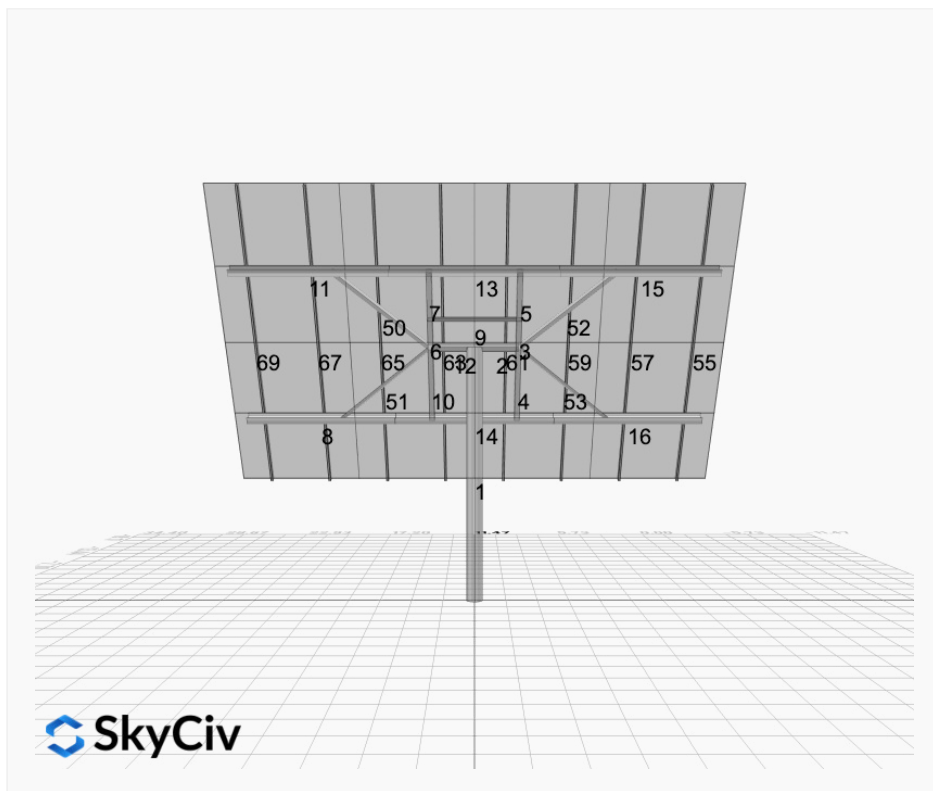
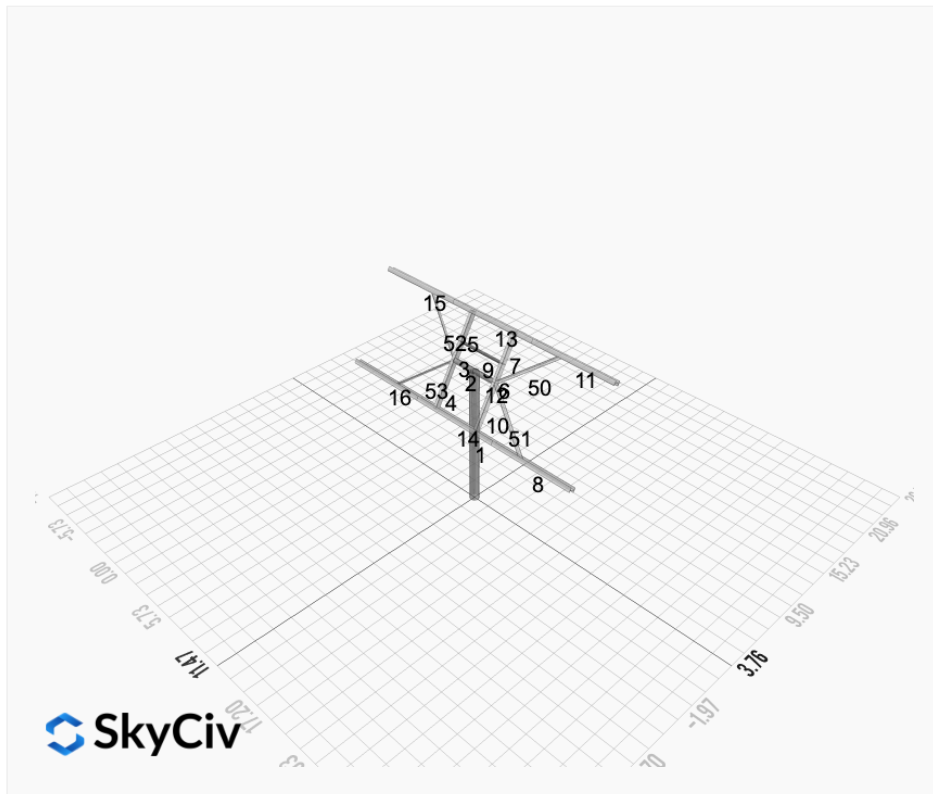
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```

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)

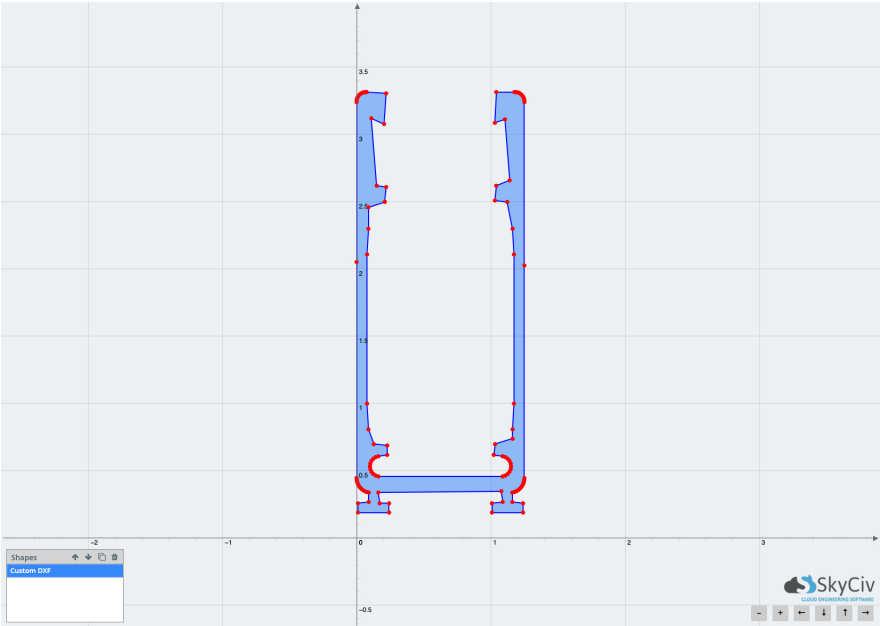




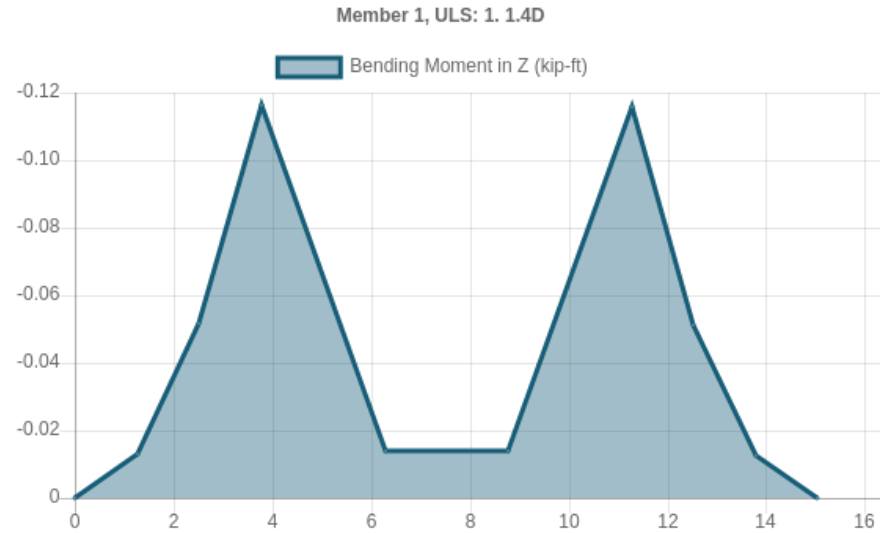


Rail Design Check

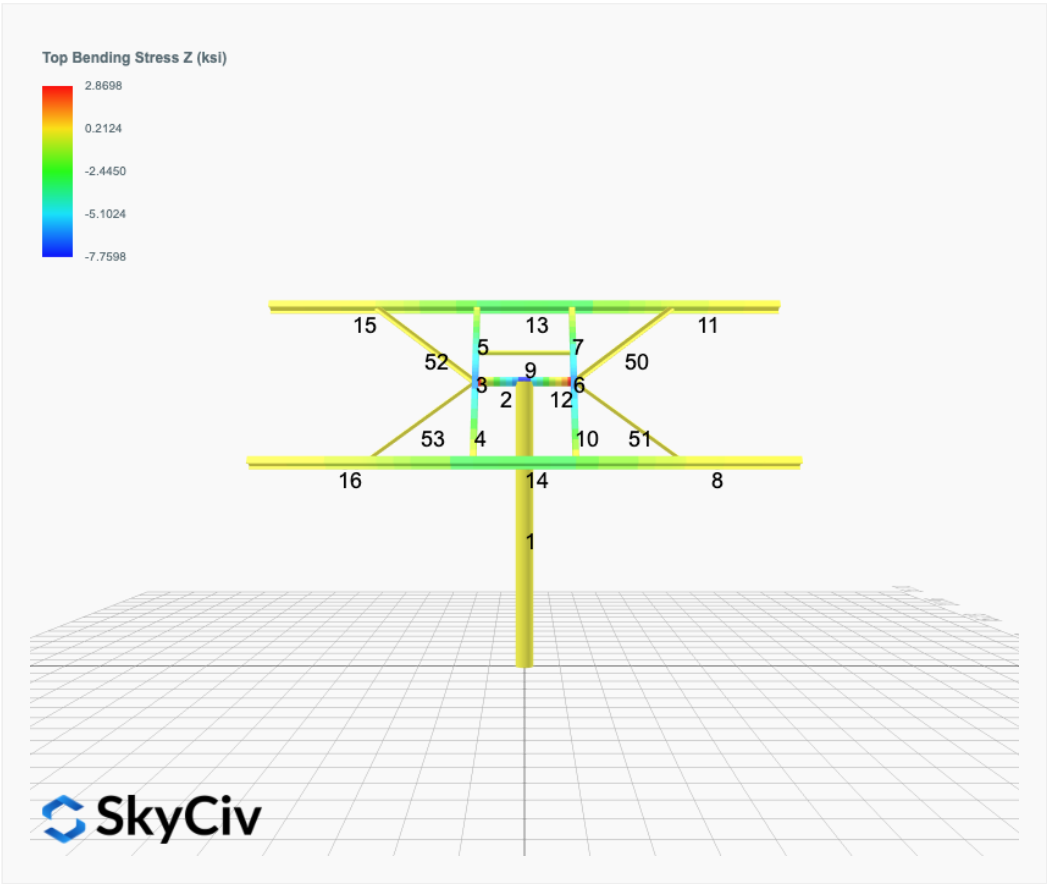
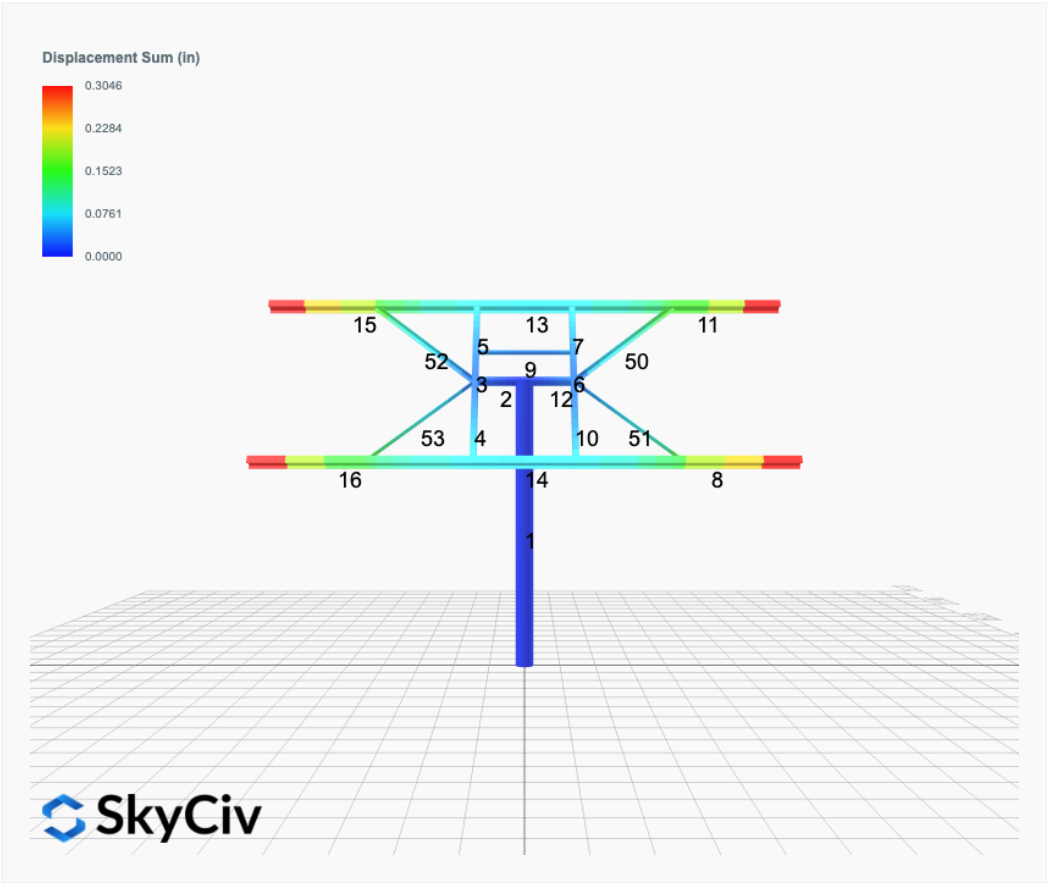
Rail Length: 15.049999999999999 ft
Additional Restraints Required: None
Tributary Width: 2.866666666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0252 kip/ft
Snow (Y): -0.0437 kip/ft
Wind uplift Case A: 0.0460 kip/ft
Wind downforce Case A: 0.0460 kip/ft
Dead (Panel load) (X): 0.0067 kip/ft
Dead (Panel load) (Y): -0.0117 kip/ft



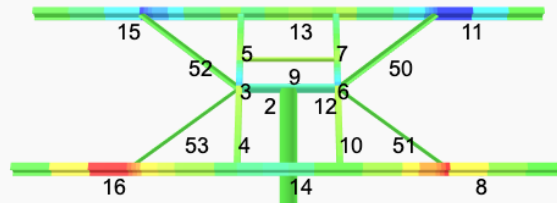
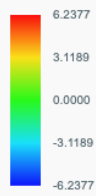
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	16.64727582	0.483	PASS
Material Yield	34.5	16.64727582	0.483	PASS
Material Strength	37	16.64727582	0.450	PASS



FEM Results (Envelope Worst Case for each member)

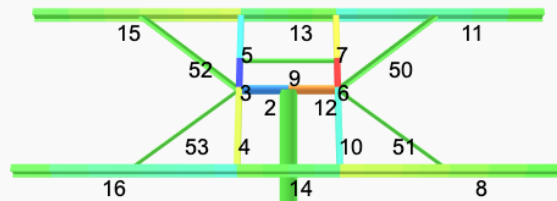


Top Bending Stress Y (ksi)



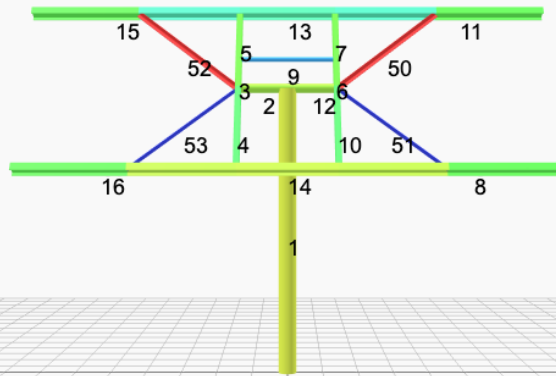
 SkyCiv

Shear Stress Y (ksi)



 SkyCiv

Axial Stress (ksi)



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	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 2. D + L	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 3. D + (S or Lr or R)	0.0000	5.4074	0.0000	-0.0000	-0.0000	0.0447
ULS: 3. D + (S or Lr or R)	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.6958	0.0000	-0.0000	-0.0000	0.0383
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 5b. D + 0.7E	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.6958	0.0000	-0.0000	-0.0000	0.0383
ULS: 8. 0.6D + 0.7E	0.0000	1.5365	0.0000	-0.0000	-0.0000	0.0115
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8805	4.2239	0.0000	-0.0000	-0.0000	33.4751
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8805	0.8978	0.0000	-0.0000	-0.0000	-32.8796
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1604	5.9431	0.0000	-0.0000	-0.0000	25.1302
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.6958	0.0000	-0.0000	-0.0000	0.0383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1604	3.4485	0.0000	-0.0000	-0.0000	-24.6358
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.6958	0.0000	-0.0000	-0.0000	0.0383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1604	3.8081	0.0000	-0.0000	-0.0000	25.1111
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1604	1.3136	0.0000	-0.0000	-0.0000	-24.6549
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.5608	0.0000	-0.0000	-0.0000	0.0192
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8805	3.1995	0.0000	-0.0000	-0.0000	33.4674
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.5365	0.0000	-0.0000	-0.0000	0.0115
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8805	-0.1265	0.0000	-0.0000	-0.0000	-32.8873
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.5365	0.0000	-0.0000	-0.0000	0.0115

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	9.0134
Shear X	-4.8009
Shear Z	0.0000
Moment X	-0.0018
Moment Y (Twist)	0.0006
Moment Z	56.7440

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	5.9431
Shear X	-2.8805
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	33.4751

Project Details

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

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

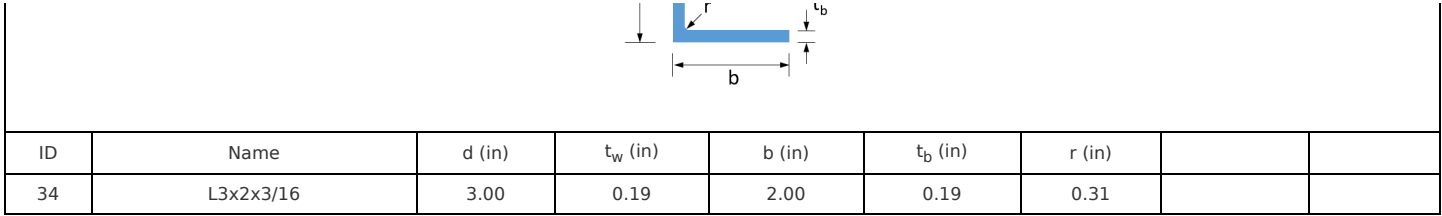


Design Input Information

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)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
9	8in Pipe Sch 40	8.63	0.32					
ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		
ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25



Member Properties												
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (f t)	C _b					LS T	LS C	LD
1	9	24.19	24.19	11.52	-					300	200	1
2	4	1.30	1.30	2.00	-					300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18					300	200	1
4	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68					300	200	1
5	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.66,1.67					300	200	1
6	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18					300	200	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.66,1.67					300	200	1
8	18	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.30,2.31,2.31,2.30,2.24,2.30,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31					300	200	1
9	1	2.60	2.60	4.00	-					300	200	1
10	15	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68					300	200	1
11	18	7.00	7.00	7.00	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.31,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.25,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.30,2.31					300	200	1
12	4	1.30	1.30	2.00	-					300	200	1
13	18	4.88	4.00	7.50	1.02,1.02					300	200	1
14	18	4.88	4.00	7.50	1.02,1.02					300	200	1
15	18	7.00	7.00	7.00	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.31,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.25,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.30,2.31					300	200	1
16	18	7.00	7.00	7.00	2.31,2.31,2.31,2.30,2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.30,2.31,2.31,2.30,2.24,2.30,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31					300	200	1
50	34	5.67	5.67	5.67	1.14,1.14					300	200	250
51	34	5.67	5.67	5.67	1.14,1.14					300	200	250
52	34	5.67	5.67	5.67	1.14,1.14					300	200	250
53	34	5.67	5.67	5.67	1.14,1.14					300	200	250

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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	185.16	83.29	83.29	113.39	113.39
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.89	10.99	6.26	29.14	16.61
4	79.65	72.84	10.99	6.26	29.14	16.61
5	79.65	74.30	10.99	6.26	29.14	16.61
6	79.65	74.89	10.99	6.26	29.14	16.61
7	79.65	74.30	10.99	6.26	29.14	16.61
8	120.60	64.33	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.84	10.99	6.26	29.14	16.61
11	120.60	64.33	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	84.02	17.94	6.45	30.09	45.74
14	120.60	84.02	17.93	6.45	30.09	45.74
15	120.60	64.33	23.36	6.45	30.09	45.74
16	120.60	64.33	23.36	6.45	30.09	45.74
50	41.27	8.45	1.63	0.76	15.23	10.15
51	41.27	8.45	1.63	0.76	15.23	10.15
52	41.27	8.45	1.63	0.76	15.23	10.15
53	41.27	8.45	1.63	0.76	15.23	10.15

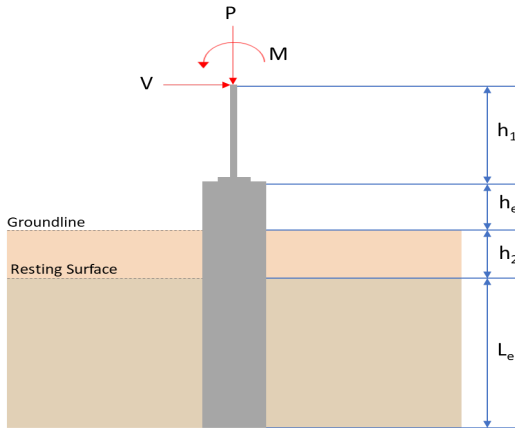
Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P, M_z , M_y)	Worst LC	KL/r	δ	Status
1	0.049	0.681	0.000	0.042	0.000	0.701	#13	0.494	Not Required	Pass
2	0.009	0.408	0.317	0.101	0.056	0.668	#13	0.052	Not Required	Pass
3	0.003	0.653	0.259	0.066	0.085	0.812	#21	0.044	Not Required	Pass
4	0.003	0.644	0.085	0.065	0.007	0.693	#13	0.078	Not Required	Pass
5	0.003	0.405	0.059	0.065	0.019	0.438	#13	0.073	Not Required	Pass
6	0.003	0.653	0.259	0.066	0.085	0.812	#21	0.044	Not Required	Pass
7	0.003	0.405	0.059	0.065	0.019	0.438	#13	0.073	Not Required	Pass
8	0.023	0.184	0.220	0.041	0.015	0.295	#21	0.464	Not Required	Pass
9	0.036	0.046	0.084	0.001	0.000	0.140	#13	0.132	Not Required	Pass
10	0.003	0.644	0.085	0.065	0.007	0.693	#13	0.078	Not Required	Pass
11	0.013	0.185	0.220	0.041	0.016	0.290	#21	0.309	Not Required	Pass
12	0.009	0.408	0.317	0.101	0.056	0.668	#13	0.052	Not Required	Pass
13	0.012	0.376	0.037	0.052	0.007	0.400	#13	0.177	Not Required	Pass
14	0.019	0.379	0.044	0.051	0.007	0.389	#13	0.265	Not Required	Pass
15	0.013	0.185	0.220	0.041	0.016	0.290	#21	0.309	Not Required	Pass
16	0.023	0.184	0.220	0.041	0.015	0.295	#21	0.309	Not Required	Pass
50	0.239	0.008	0.008	0.002	0.002	0.253	#21	0.783	Not Required	Pass
51	0.048	0.003	0.016	0.001	0.001	0.062	#6	0.522	Not Required	Pass
52	0.239	0.008	0.008	0.002	0.002	0.253	#23	0.783	Not Required	Pass
53	0.048	0.003	0.017	0.001	0.001	0.062	#23	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/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 = 6.75 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>5.943</td><td>9.013</td></tr><tr><td>Vx (kip)</td><td>-2.881</td><td>-4.801</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>-0.002</td></tr><tr><td>Mz (kipft)</td><td>33.475</td><td>56.744</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)	5.943	9.013	Vx (kip)	-2.881	-4.801	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	-0.002	Mz (kipft)	33.475	56.744	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, $H_o = \frac{V_x}{1.57 \text{ } D}$ $H_o = \frac{(-2.881 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.45876 \text{ kip/ft}$	
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)	-2.881	-4.801																											
Vz (kip)	0.000	0.000																											
Mx (kipft)	0.000	-0.002																											
Mz (kipft)	33.475	56.744																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(33.475 \text{ kipft}) + ((-2.881 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 5.3304 \text{ kipft/ft}$ 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: $L_{e,x} = 6.3202 \text{ ft}$ - 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, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(6.3202 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.32 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6.75 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(6.32 \text{ ft})}{(6.75 \text{ ft})}$ $Ratio = 0.9363$	Status: PASS Ratio: 0.940
	End-bearing Capacity (ASD) A - Pile cross-section area $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ q - End-bearing pressure $q = \frac{P_v}{A}$ $q = \frac{(5.943 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.37144 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.37144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.18572$	Status: PASS Ratio: 0.190
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.45876 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.3304 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.3304 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.45876 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.3304 \text{ kipft/ft})) + (4 \times (-0.45876 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.657 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.3304 \text{ kipft/ft})) + (3 \times (-0.45876 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (5.3304 \text{ kipft/ft})) + (2 \times (-0.45876 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.24321 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.3304 \text{ kipft/ft})) + ((-0.45876 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.99611 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.657 \text{ ft})}{2}$$

$$p_a = 0.34928 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24321 \text{ kip/ft}^2)}{(0.34928 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.69632$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

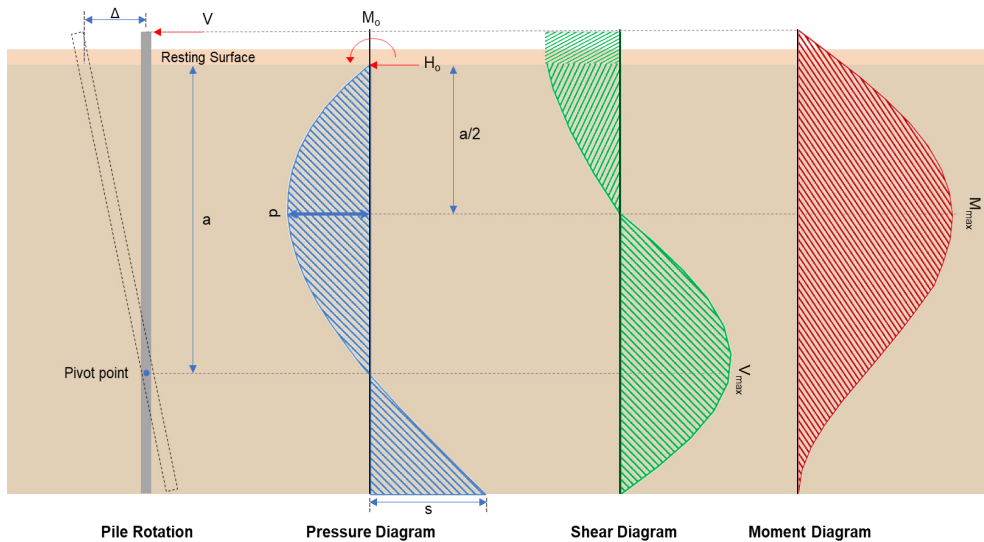
$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.99611 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.700**

**Shear force and Bending moment (x-direction, LRFD)** H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.801 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.76449 \text{ kip/ft}$$

 M_o - Moment per length of pile,

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

$$M_o = \frac{(56.744 \text{ kipft}) + ((-4.801 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.0357 \text{ kipft/ft}$$

 E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.0357 \text{ kipft/ft})}{(-0.76449 \text{ kip/ft})}$$

$$E = 11.819 \text{ ft}$$

 a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.0357 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.76449 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (9.0357 \text{ kipft/ft})) + (4 \times (-0.76449 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6551 \text{ ft}$$

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

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

$$V_{max} = ((-0.76449 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.819 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.819 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 11.492 \text{ kip}$ M_{max} - Max bending moment located at depth a/2, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 \right] + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right]$ $M_{max} = ((-0.76449 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.819 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.819 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.819 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$ $M_{max} = 36.91 \text{ kipft}$	
	Shear force and Bending moment (z-direction, LRFD) H_o - Lateral force per length of pile, $H_o = \frac{V_z}{1.57 b}$ $H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0 \text{ kip/ft}$ M_o - Moment per length of pile, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.00031847 \text{ kipft/ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (6.75 \text{ ft}))}$ $a = 4.5 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$ $V_{max} = 12 \times \left(\frac{(0.00031847 \text{ kipft/ft}) \times (48 \text{ in})}{(6.75 \text{ ft})} \right) \times \left(\frac{(4.5 \text{ ft})}{(6.75 \text{ ft})} - 1 \right) \times \left(\frac{(4.5 \text{ ft})}{(6.75 \text{ ft})} \right)^2$ $V_{max} = 0.00033551 \text{ kip}$ M_{max} - Max bending moment at depth a/2, $M_{max} = (M_o b) \left[1 - \left(4 \frac{a}{2 L_e} \right)^3 + \left(3 \frac{a}{2 L_e} \right)^4 \right]$ $M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.5 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 + \left(3 \times \frac{(4.5 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right]$ $M_{max} = 0.0011323 \text{ kipft}$	
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, $A_{st,required}$ $A_{st,required}$	

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

$$A_{st,required} = Min \left[\frac{\frac{(9.013 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.297 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max[(-84.297 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$$

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10: 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**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 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

	$Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.013 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0033691$	Status: PASS Ratio: 0.000
	Shear Strength (ACI 318-19, LRFD) Parameters: 22.5.2.2 $b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.013 \text{ kip} \rightarrow 9013 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9013 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.69 \text{ kip}$ 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 $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(296.21 \text{ kip}), (119.69 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.69 \text{ kip}$ 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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.69 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.88 \text{ kip}$ Considering x-direction: $V_{max} = 11.492 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(11.492 \text{ kip})}{(110.88 \text{ kip})}$ $Ratio = 0.10364$		Status: PASS Ratio: 0.100
	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: $\phi M_{n,1}$ $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ $\phi M_{n,2}$ $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$		
14.5.2.1b			

	Therefore, ϕM_n - Allowable flexural strength, $\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ Considering x-direction: $M_{max} = 36.91 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(36.91 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.14788$	Status: PASS Ratio: 0.150
	Considering z-direction: $M_{max} = 0.0011323 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(0.0011323 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 4.5366 \times 10^{-6}$	Status: PASS Ratio: 0.000