

Project Details



Project Name: DjsCabinTOP-16

Date: Thu Jul 24 2025

Location: Hughes Creek, Montana, USA

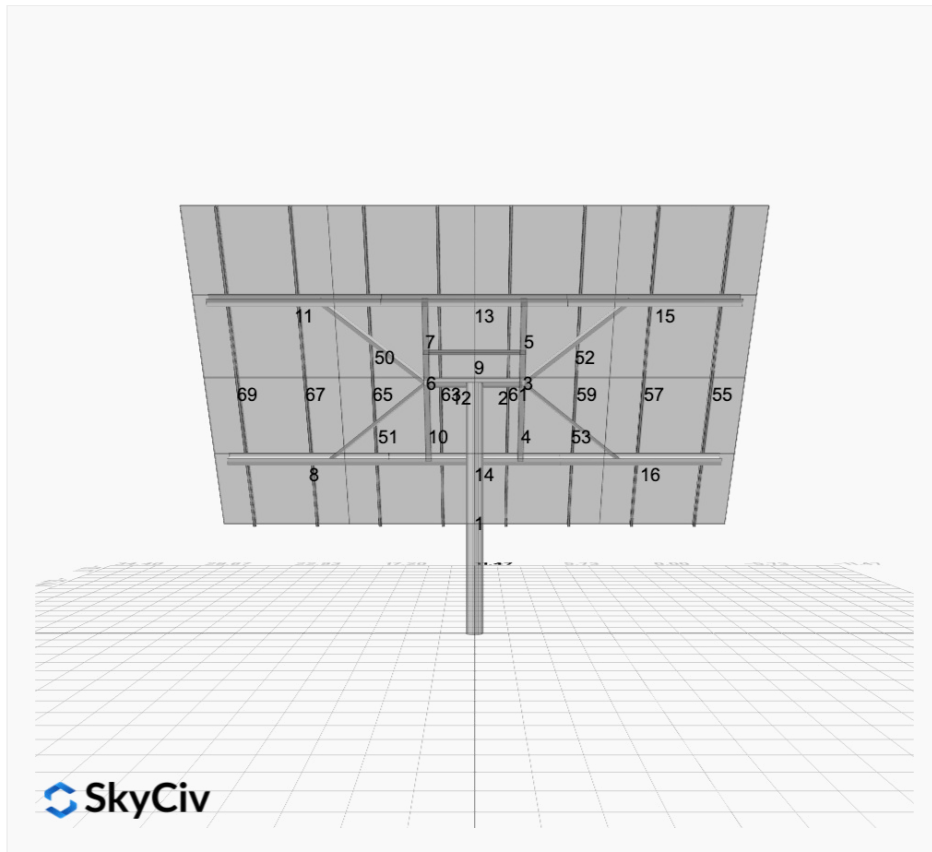
Number of Modules: 16

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

Number of Poles: 1

Date Sold:

Dealer: _____



Array Dimensions N/S	15.05 ft
Array Dimensions E/W	22.93 ft
Winter Tilt Angle	60
Front Edge Clearance	4 ft

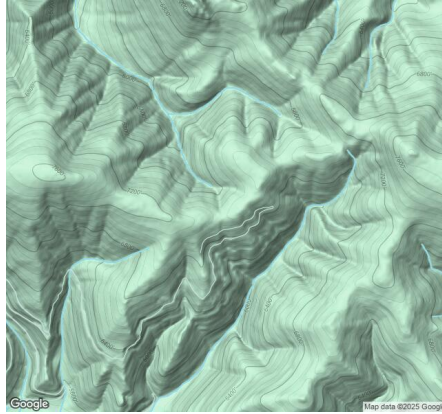
MT Solar Bill of Materials (1P-0-8TOP-SD-84-L-4Hx4W-STRUTS-0AGG)

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: Hughes Creek, Montana, 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:	4
Total Array Height at Tilt:	17.03 ft
Total Frame Length:	21.50 ft
Module Info/Notes:	JA Solar 405
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:	10.52 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: 6.00 ft
Foundation Volume:	3.556 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Hughes Creek, Montana, USA
Wind Speed:	99 mph
Snow Load:	50 psf

Design Disclaimer

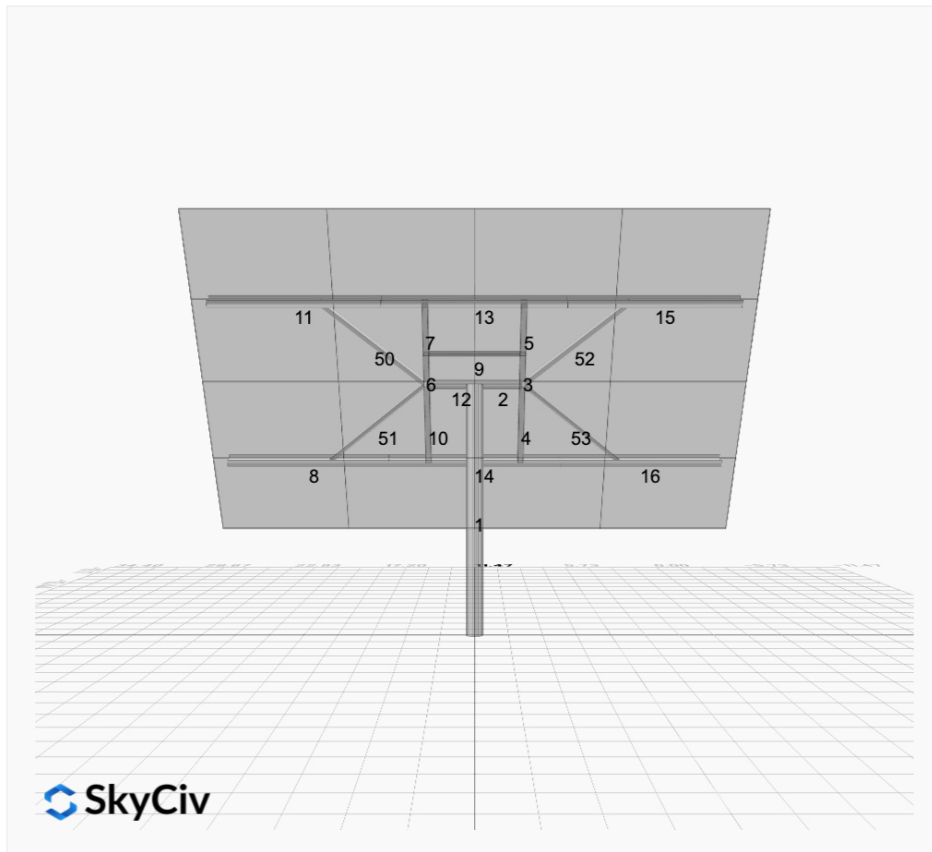
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

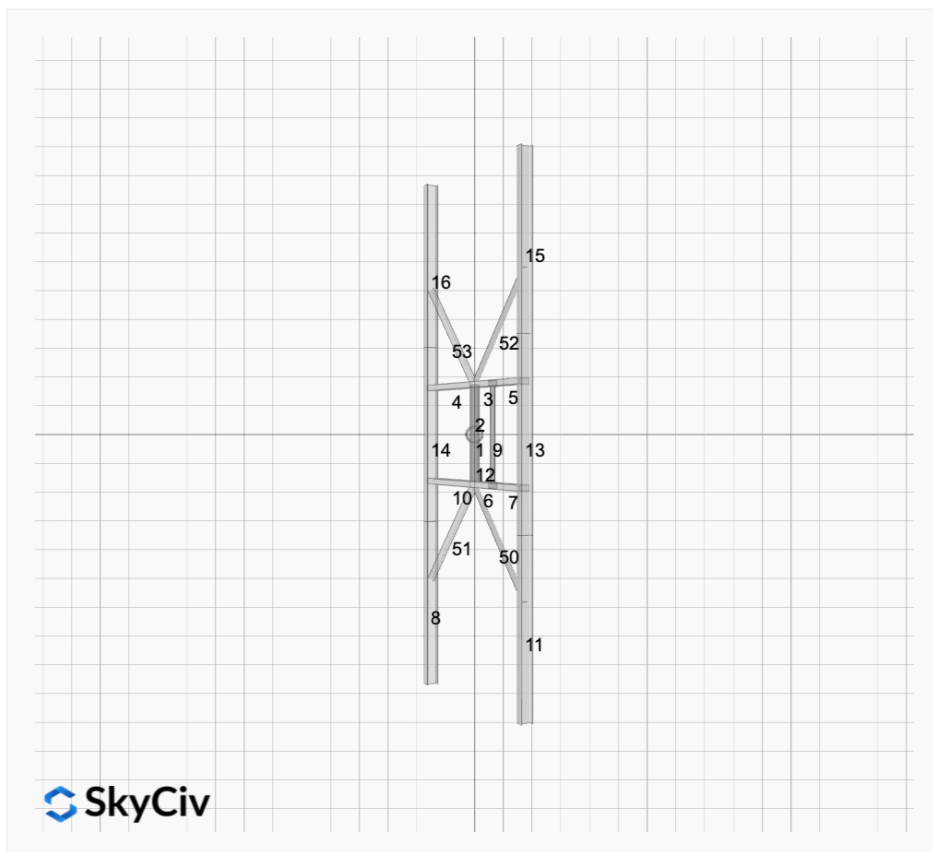
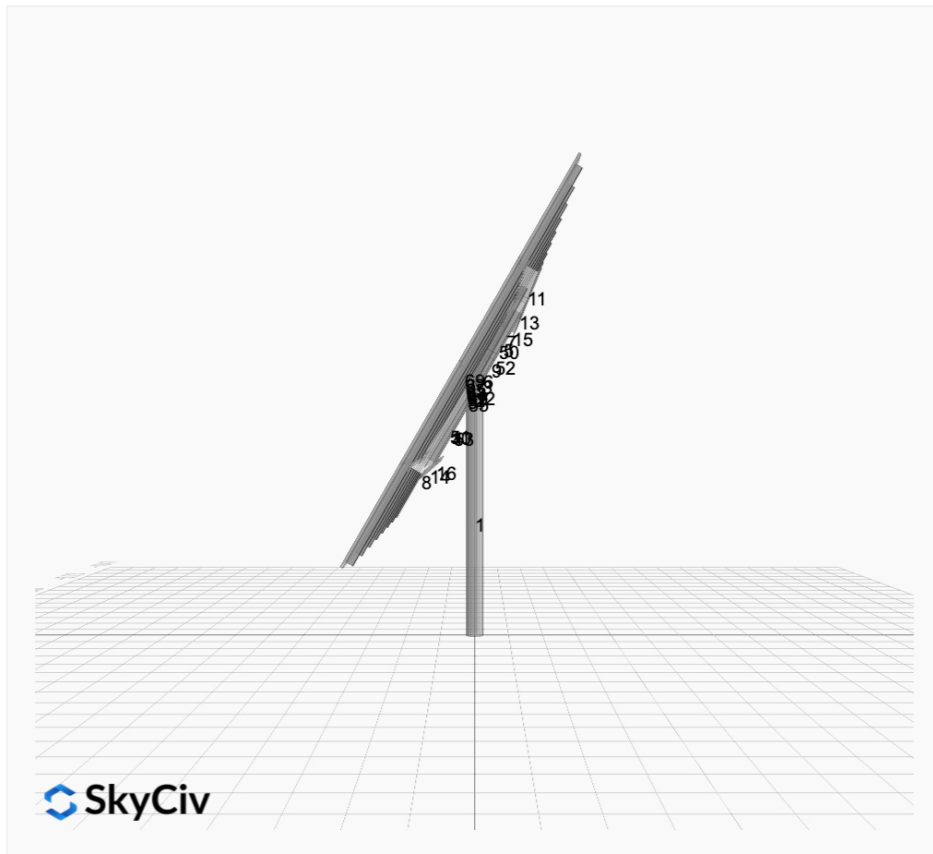
AutoDesigner Input

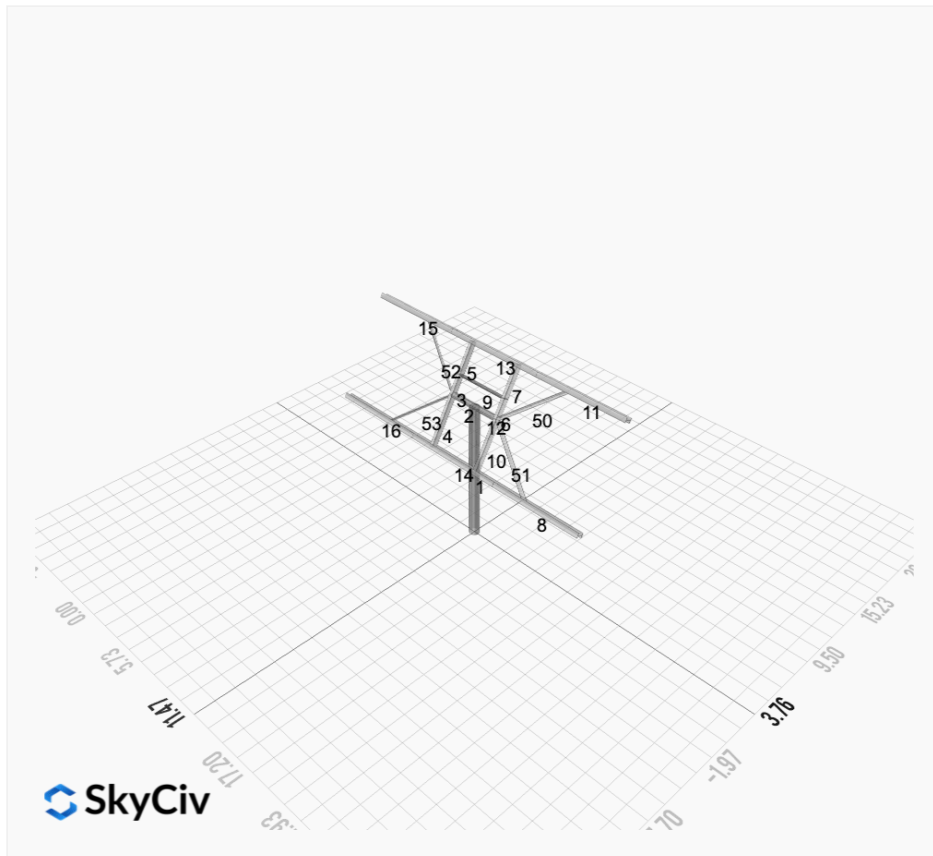
```
{
  "product_type": "Beam",
  "designer_name": "Jordan Solar",
  "designer_email": "janelle@jordansolar.net",
  "designer_phone": "4062065099",
  "project_id": "DJsCabinTOP-16",
  "site_address": "Hughes Creek, Montana, USA",
  "module_info": "JA Solar 405",
  "module_width": 44.65,
  "module_length": 67.8,
  "number_rows": 4,
  "number_columns": 4,
  "pole_mount_section": "4_40",
  "core_pipe_width": 65,
  "core_pipe_section": "2_40",
  "adjuster_section": "2_40",
  "core_beam_height": 65,
  "core_beam_section": "HSS3x2x1/8",
  "main_pipe_section": "2_12GA",
  "pole_spacing": 15,
  "tilt_angle": 60,
  "ground_clearance": 4,
  "risk_category": "I",
  "exposure_category": "B",
  "frame_duty_override": "auto",
  "pole_override": "8_40",
  "soil_type": "sand",
  "customer_foundation_override": "48_Square",
  "foundation_type": "Square",
  "foundation_size": 48,
  "check_rails": true,
  "wind_speed_override": 99,
  "snow_load_override": 50,
  "direct_snow_load": false,
  "add_angle_brace": true
}
```

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)

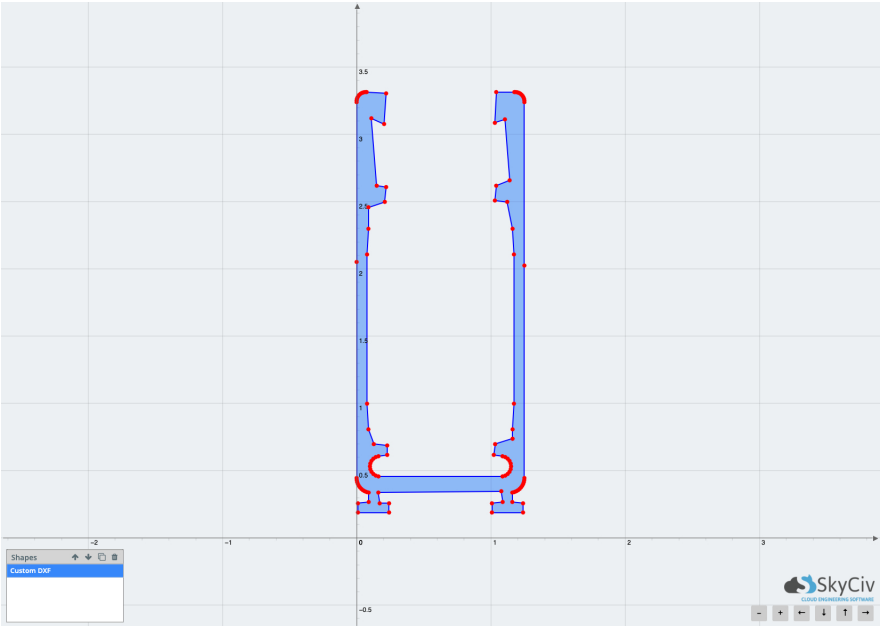






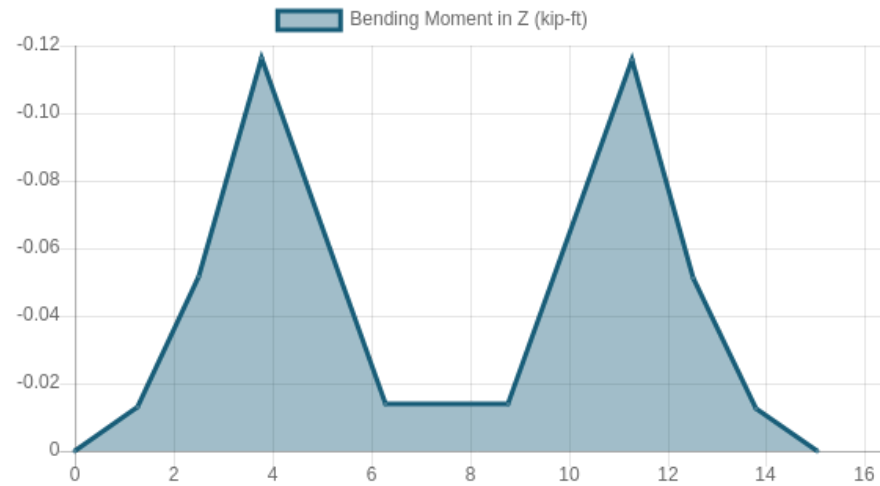
Rail Design Check

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): 0.0079 kip/ft
Snow (Y): -0.0136 kip/ft
Wind uplift Case A: 0.0350 kip/ft
Wind downforce Case A: 0.0350 kip/ft
Dead (Panel load) (X): 0.0067 kip/ft
Dead (Panel load) (Y): -0.0117 kip/ft

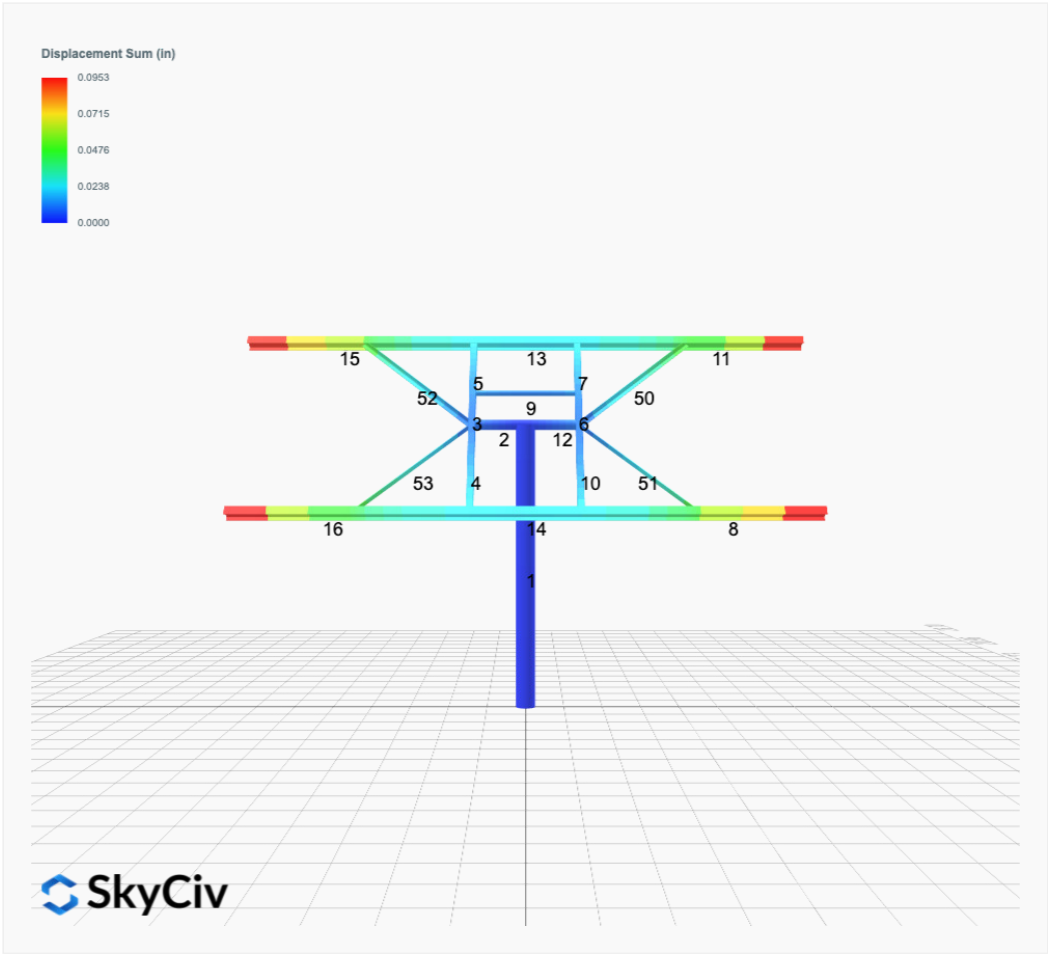


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	8.62700199	0.250	PASS
Material Yield	34.5	8.62700199	0.250	PASS
Material Strength	37	8.62700199	0.233	PASS

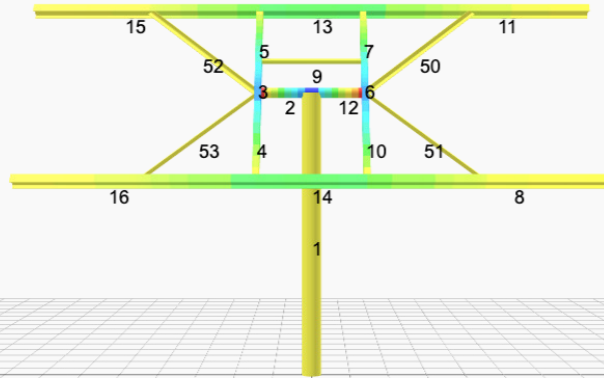
Member 1, ULS: 1.14D



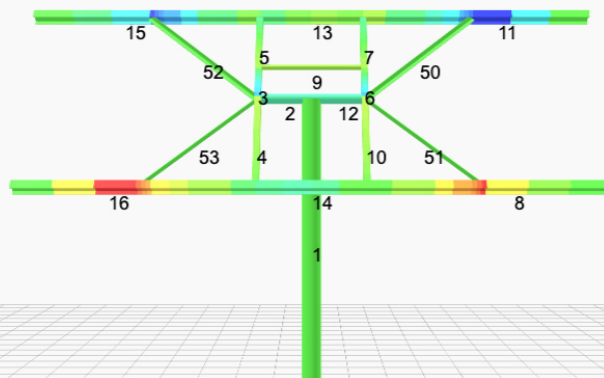
FEM Results (Envelope Worst Case for each member)

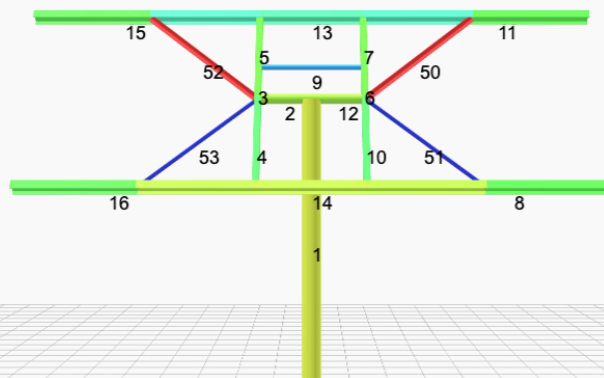
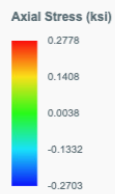
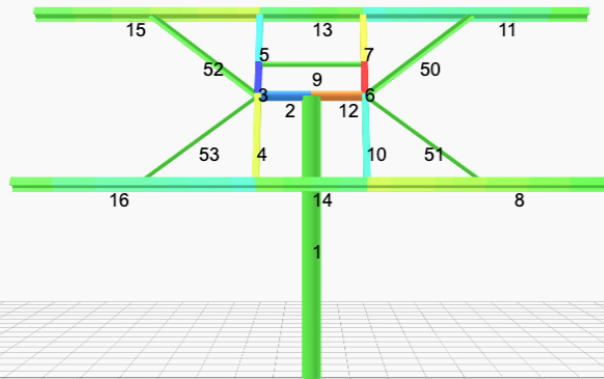
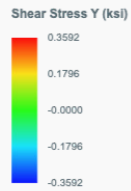


Top Bending Stress Z (ksi)



Top Bending Stress Y (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	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 2. D + L	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 3. D + (S or Lr or R)	0.0000	3.4218	0.0000	-0.0000	-0.0000	0.0217
ULS: 3. D + (S or Lr or R)	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.1994	0.0000	-0.0000	-0.0000	0.0211
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 5b. D + 0.7E	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.1994	0.0000	-0.0000	-0.0000	0.0211
ULS: 8. 0.6D + 0.7E	0.0000	1.5194	0.0000	-0.0000	-0.0000	0.0115
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.1890	3.7960	0.0000	-0.0000	-0.0000	23.1634
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1890	1.2685	0.0000	-0.0000	-0.0000	-22.8801
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.5323	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	-1.6417	4.1473	0.0000	-0.0000	-0.0000	17.3792
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	3.1994	0.0000	-0.0000	-0.0000	0.0211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6417	2.2516	0.0000	-0.0000	-0.0000	-17.1534
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	3.1994	0.0000	-0.0000	-0.0000	0.0211
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6417	3.4801	0.0000	-0.0000	-0.0000	17.3774
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.5323	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	1.6417	1.5844	0.0000	-0.0000	-0.0000	-17.1552
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.5323	0.0000	-0.0000	-0.0000	0.0192
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1890	2.7831	0.0000	-0.0000	-0.0000	23.1557
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.5194	0.0000	-0.0000	-0.0000	0.0115
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1890	0.2556	0.0000	-0.0000	-0.0000	-22.8877
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.5194	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	5.5898
Shear X	-3.6483
Shear Z	0.0000
Moment X	-0.0011
Moment Y (Twist)	0.0004
Moment Z	39.0382

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	4.1473
Shear X	-2.1890
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	23.1634

Project Details

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

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

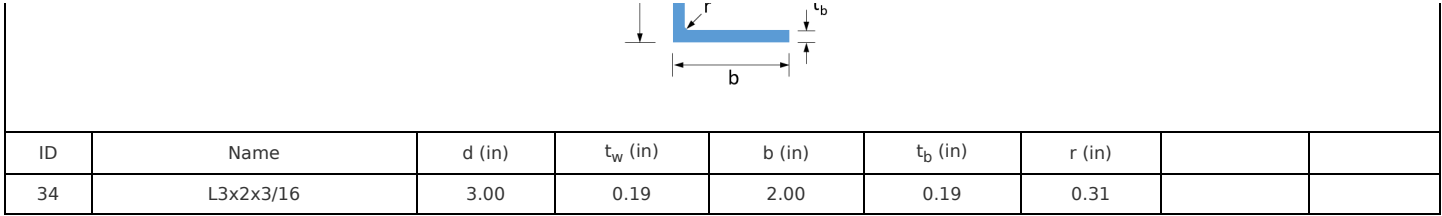


Design Input Information

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	22.0 9	22.0 9	10. 52	-	30 0	20 0	1
2	4	1.30	1.30	2.0 0	-	30 0	20 0	1
3	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	30 0	20 0	1
4	15	2.44	2.44	3.7 5	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.59,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	30 0	20 0	1
5	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.62,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	30 0	20 0	1
6	15	0.92	0.92	1.4 2	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.14,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	30 0	20 0	1
7	15	1.52	1.52	2.3 3	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.62,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	30 0	20 0	1
8	18	7.00	7.00	7.0 0	2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.29,2.31,2.31,2.31,2.29,2.31,2.30,2.31,2.30,2.31	30 0	20 0	1
9	1	2.60	2.60	4.0 0	-	30 0	20 0	1
10	15	2.44	2.44	3.7 5	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.59,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	30 0	20 0	1
11	18	7.00	7.00	7.0 0	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.30,2.31,2.31,2.32,2.29,2.32,2.31,2.31,2.30,2.31	30 0	20 0	1
12	4	1.30	1.30	2.0 0	-	30 0	20 0	1
13	18	4.88	4.00	7.5 0	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.05,1.02,1.02,1.02,1.03,1.02,1.02,1.02,1.02,1.02	30 0	20 0	1
14	18	4.88	4.00	7.5 0	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.04,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02	30 0	20 0	1
15	18	7.00	7.00	7.0 0	2.32,2.31,2.32,2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.31,2.32,2.30,2.32,2.31,2.31,2.29,2.31,2.31,2.32,2.29,2.32,2.31,2.31,2.30,2.31	30 0	20 0	1
16	18	7.00	7.00	7.0 0	2.31,2.31,2.31,2.31,2.31,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.30,2.31,2.29,2.31,2.31,2.31,2.29,2.31,2.30,2.31,2.30,2.31	30 0	20 0	1
50	34	5.67	5.67	5.6 7	1.14,1.14	30 0	20 0	25 0
51	34	5.67	5.67	5.6 7	1.14,1.14	30 0	20 0	25 0
52	34	5.67	5.67	5.6 7	1.14,1.14	30 0	20 0	25 0
53	34	5.67	5.67	5.6 7	1.14,1.14	30 0	20 0	25 0

SkyCiv AutoDesigner Report - DJS CabinTOP-16
Page 13 of 24

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	208.46	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

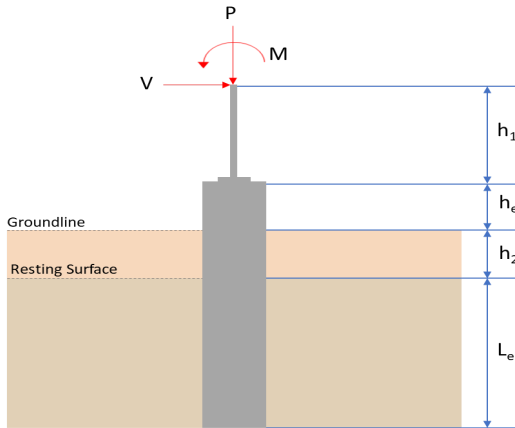
Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P, M_z , M_y)	Worst LC	KL/r	δ	Status
1	0.027	0.469	0.000	0.032	0.000	0.482	#13	0.451	Not Required	Pass
2	0.005	0.264	0.241	0.061	0.043	0.507	#13	0.052	Not Required	Pass
3	0.002	0.492	0.146	0.050	0.048	0.604	#13	0.044	Not Required	Pass
4	0.002	0.487	0.049	0.049	0.004	0.524	#13	0.078	Not Required	Pass
5	0.002	0.305	0.032	0.049	0.010	0.330	#13	0.073	Not Required	Pass
6	0.002	0.492	0.146	0.050	0.048	0.604	#13	0.044	Not Required	Pass
7	0.002	0.305	0.032	0.049	0.010	0.330	#13	0.073	Not Required	Pass
8	0.013	0.139	0.121	0.031	0.008	0.168	#21	0.464	Not Required	Pass
9	0.020	0.035	0.063	0.001	0.000	0.106	#13	0.132	Not Required	Pass
10	0.002	0.487	0.049	0.049	0.004	0.524	#13	0.078	Not Required	Pass
11	0.007	0.139	0.121	0.031	0.009	0.165	#21	0.309	Not Required	Pass
12	0.005	0.265	0.241	0.061	0.043	0.507	#13	0.052	Not Required	Pass
13	0.007	0.283	0.020	0.039	0.004	0.301	#13	0.177	Not Required	Pass
14	0.010	0.286	0.025	0.038	0.004	0.294	#13	0.265	Not Required	Pass
15	0.007	0.139	0.121	0.031	0.009	0.165	#21	0.309	Not Required	Pass
16	0.013	0.139	0.121	0.031	0.008	0.168	#21	0.309	Not Required	Pass
50	0.132	0.008	0.008	0.001	0.001	0.146	#21	0.783	Not Required	Pass
51	0.026	0.003	0.016	0.000	0.001	0.042	#6	0.522	Not Required	Pass
52	0.132	0.008	0.008	0.001	0.001	0.146	#23	0.783	Not Required	Pass
53	0.026	0.003	0.017	0.000	0.001	0.042	#6	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 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><thead><tr><th>Layer</th><th>Label</th><th>Allowable Bearing Pressure (qa) (psf)</th><th>Allowable Lateral Pressure (R) (psf/ft)</th></tr></thead><tbody><tr><td>1</td><td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td><td>2000.000</td><td>150.000</td></tr></tbody></table> Tabulation of Loads <table><thead><tr><th>Load Component</th><th>ASD</th><th>LRFD</th></tr></thead><tbody><tr><td>P (kip)</td><td>4.147</td><td>5.590</td></tr><tr><td>Vx (kip)</td><td>-2.189</td><td>-3.648</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>-0.001</td></tr><tr><td>Mz (kipft)</td><td>23.163</td><td>39.038</td></tr></tbody></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)	4.147	5.590	Vx (kip)	-2.189	-3.648	Vz (kip)	0.000	0.000	Mx (kipft)	0.000	-0.001	Mz (kipft)	23.163	39.038	
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)	4.147	5.590																										
Vx (kip)	-2.189	-3.648																										
Vz (kip)	0.000	0.000																										
Mx (kipft)	0.000	-0.001																										
Mz (kipft)	23.163	39.038																										
	Required depth to resist lateral loads (ASD) H - Point of application of the lateral load H = h1 + h2 + he H = (0 ft) + (0 ft) + (0 ft) H = 0 ft Considering x-direction: Ho - Lateral force per length of pile, Ho = Vx / 1.57 D Ho = (-2.189 kip) / 1.57 x (48 in) Ho = -0.34857 kip/ft																											

	M_o - Moment per length of pile, $M_o = \frac{M_z + (V_x H)}{1.57 D}$ $M_o = \frac{(23.163 \text{ kipft}) + ((-2.189 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.6884 \text{ kipft/ft}$ 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: $L_{e,x} = 5.6202 \text{ ft}$ - 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, $L_{e,req} = MAX[L_{e,x}, L_{e,z}]$ $L_{e,req} = MAX[(5.6202 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 5.62 \text{ ft}$ L_e - Actual embedded length of pile, $L_e = L - h_e - h_2$ $L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 6 \text{ ft}$ Ratio - Embedded depth $Ratio = \frac{L_{e,req}}{L_e}$ $Ratio = \frac{(5.62 \text{ ft})}{(6 \text{ ft})}$ $Ratio = 0.93667$	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 $q = \frac{P_v}{A}$ $q = \frac{(4.147 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.25919 \text{ kip/ft}^2$ Check bearing capacity ratio: Ratio - Capacity $Ratio = \frac{q}{q_a}$ $Ratio = \frac{(0.25919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $Ratio = 0.12959$	Status: PASS Ratio: 0.130
Czerniak	Lateral Soil Pressure (ASD): L/D - Length to least lateral dimension ratio,	

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.34857 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.6884 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.6884 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.34857 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (3.6884 \text{ kipft/ft})) + (4 \times (-0.34857 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1372 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6884 \text{ kipft/ft})) + (3 \times (-0.34857 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (3.6884 \text{ kipft/ft})) + (2 \times (-0.34857 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.21764 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.6884 \text{ kipft/ft})) + ((-0.34857 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.88089 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1372 \text{ ft})}{2}$$

$$p_a = 0.31029 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21764 \text{ kip/ft}^2)}{(0.31029 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.70142$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

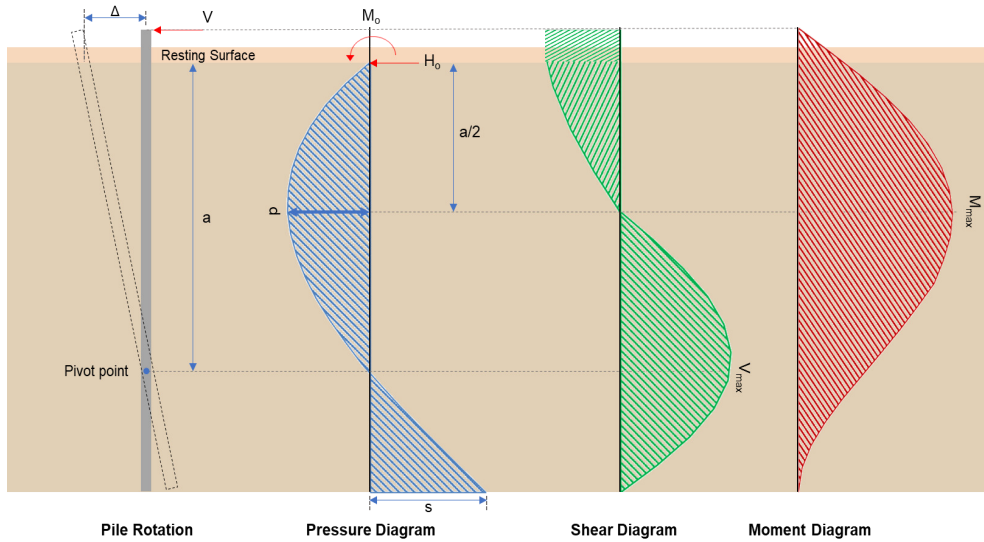
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.88089 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

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

$$Ratio = 0.97877$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.648 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.038 \text{ kipft}) + ((-3.648 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.2162 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.2162 \text{ kipft/ft})}{(-0.58089 \text{ kip/ft})}$$

$$E = 10.701 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2162 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.58089 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.2162 \text{ kipft/ft})) + (4 \times (-0.58089 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.136 \text{ ft}$$

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

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

$$V_{max} = ((-0.58089 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.701 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.136 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.701 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.136 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

	$V_{max} = 0.0059 \text{ kip}$ M_{max} - Max bending moment located at depth a/2, $M_{max} = (H_o D L_e) \left[\left(\frac{E}{L_e} + \frac{a}{2 L_e} \right) - \left[\left(\frac{4 E}{L_e} + 3 \right) \left(\frac{a}{2 L_e} \right)^3 + \left[\left(\frac{3 E}{L_e} + 2 \right) \left(\frac{a}{2 L_e} \right)^4 \right] \right] \right]$ $M_{max} = ((-0.58089 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(10.701 \text{ ft})}{(6 \text{ ft})} + \frac{(4.136 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.701 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.136 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (10.701 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.136 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right] \right]$ $M_{max} = 25.331 \text{ kipft}$	
	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, $M_o = \frac{M_x + (V_z H)}{1.57 b}$ $M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 0.00015924 \text{ kipft/ft}$ a - Distance from resting surface to pivot point, $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4 \text{ ft}$ V_{max} - Max shear force located at depth a, $V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$ $V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(6 \text{ ft})} \right) \times \left(\frac{(4 \text{ ft})}{(6 \text{ ft})} - 1 \right) \times \left(\frac{(4 \text{ ft})}{(6 \text{ ft})} \right)^2$ $V_{max} = 0.00018872 \text{ kip}$ M_{max} - Max bending moment at depth a/2, $M_{max} = (M_o b) \left[1 - \left(4 \frac{a}{2 L_e} \right)^3 + \left(3 \frac{a}{2 L_e} \right)^4 \right]$ $M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left(3 \times \frac{(4 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right]$ $M_{max} = 0.00056617 \text{ kipft}$	
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, $A_{st,required}$ $A_{st,required}$	

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

$$A_{st,required} = Min \left[\frac{\frac{(5.59 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.41 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max[(-84.41 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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,

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

$$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$$

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

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

	$Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(5.59 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0020896$	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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.59 \text{ kip} \rightarrow 5590 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(5590 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.23 \text{ kip}$ 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 $V_c = MIN [V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = MIN [(296.21 \text{ kip}), (119.23 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.23 \text{ kip}$ 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 $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.23 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.58 \text{ kip}$ Considering x-direction: $V_{max} = 8.8659 \text{ kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(8.8659 \text{ kip})}{(110.58 \text{ kip})}$ $Ratio = 0.080176$		Status: PASS Ratio: 0.080
	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: $M_{max} = 25.331 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(25.331 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 0.10149$	Status: PASS Ratio: 0.100
	Considering z-direction: $M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $Ratio = \frac{M_{max}}{\phi M_n}$ $Ratio = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$ $Ratio = 2.2683 \times 10^{-6}$	Status: PASS Ratio: 0.000