Project Details

Project Name: MTSOLAR_G16I02J4H2BA-6ft foundation-2

Location: 20 Sherry Rd, Harvard, MA 01451, USA

Unique ID: 1P-0-8TOP-HD-57-L-5Hx3W-A0E6

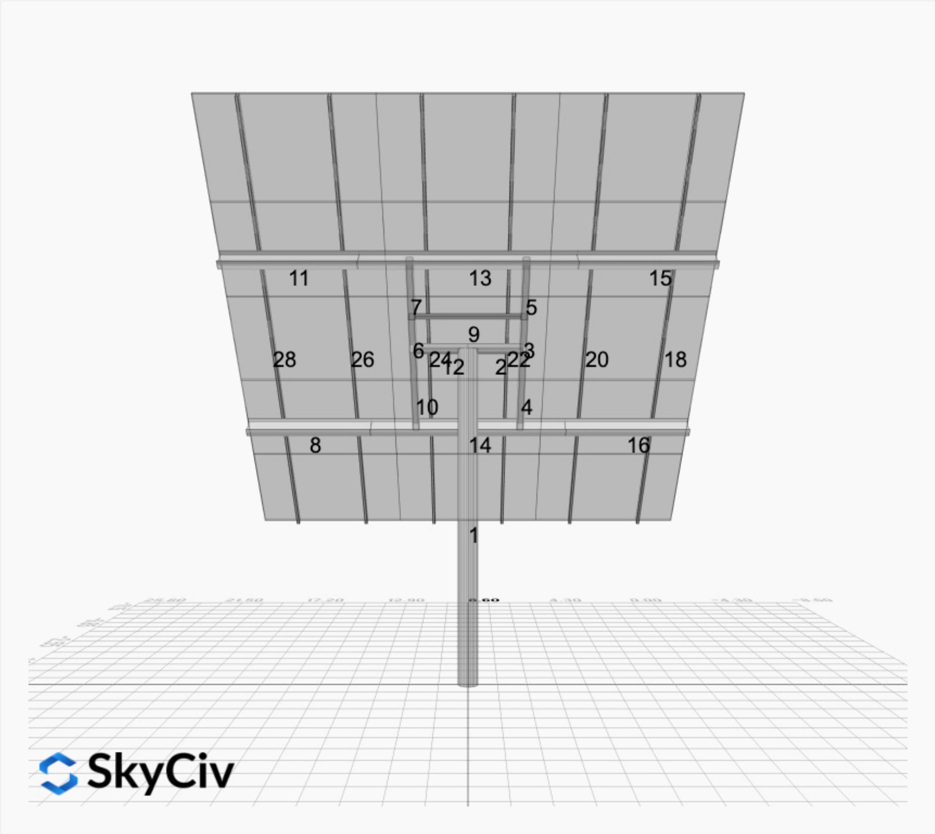
Dealer: ______________________________

Date: Thu Nov 06 2025

Number of Modules: 15

Number of Poles: 1

Date Sold: ______________________________



Array Dimensions N/S	18.83 ft
Array Dimensions E/W	17.20 ft
Winter Tilt Angle (Degrees)	50
Front Edge Clearance	5

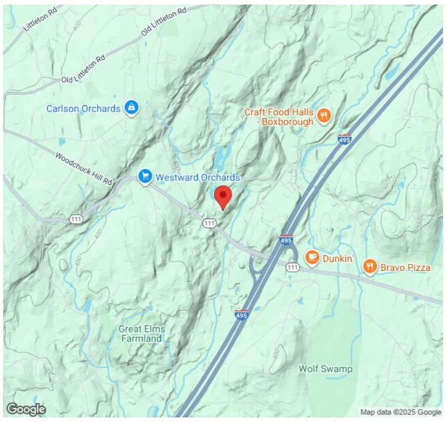
MT Solar Bill of Materials (1P-0-8TOP-HD-57-L-5Hx3W-A0E6)

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

Rail Bill of Materials

Part	Qty
Rails (226in Long)	6x
Rail Attachment	24x
Module Mid Clamp	24x
Module End Clamp	12x
Ground Lug	3x

Site Details:



Site Address: 20 Sherry Rd, Harvard, MA 01451, USA

Array Specifications

Duty Classification:	HD
Module Width:	44.70 in
Module Length:	67.80 in
Number of Rows:	5
Number of Columns:	3
Total Number of Modules:	15
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	19.43 ft
Total Frame Length:	17.00 ft
Module Info/Notes:	Telesun Bifacial 450
Array Dimensions N/S:	18.83 ft
Array Dimensions E/W:	17.20 ft
Rail Length:	226.00 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	12.21 ft
Number of Poles:	1
Pole Spacing:	0

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	20 Sherry Rd, Harvard, MA 01451, USA
Wind Speed:	116 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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{"wind_speed_override":116,"snow_load_override":50,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"MTSOLAR_G16I02J4H2BA-6ft foundation-2","site_address":"20 Sherry Rd, Harvard, MA 01451, USA","module_info":"Telesun Bifacial 450","module_width":44.7,"module_length":67.8,"number_rows":"5","number_columns":3,"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":50,"ground_clearance":5,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":84,"check_rails":false}
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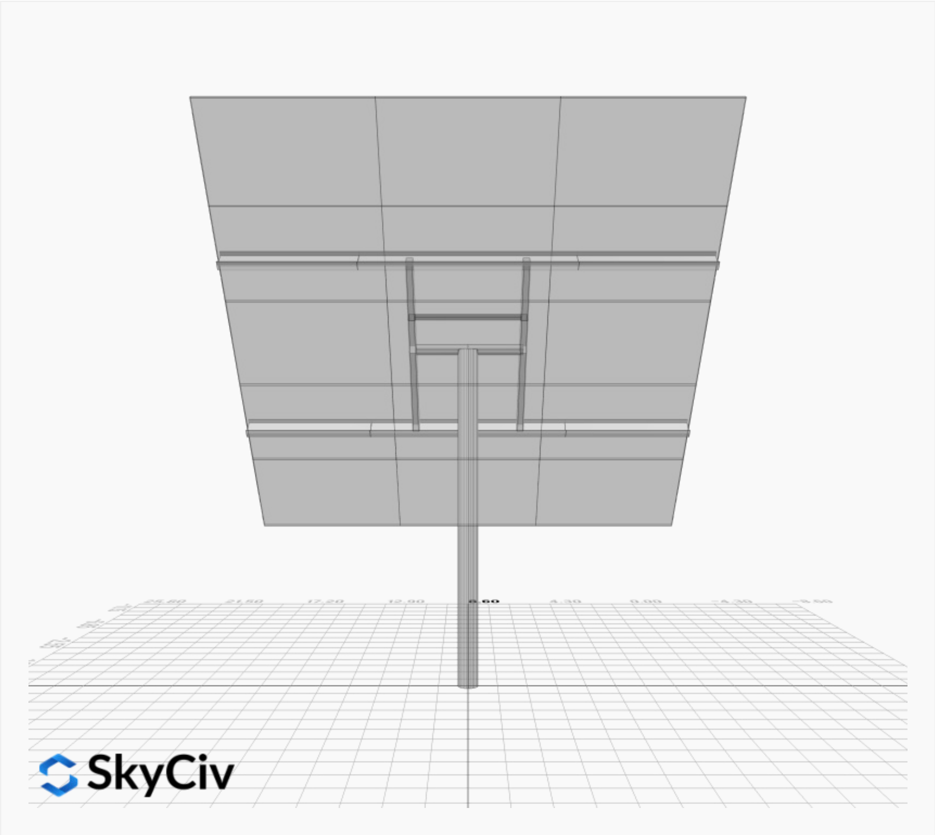
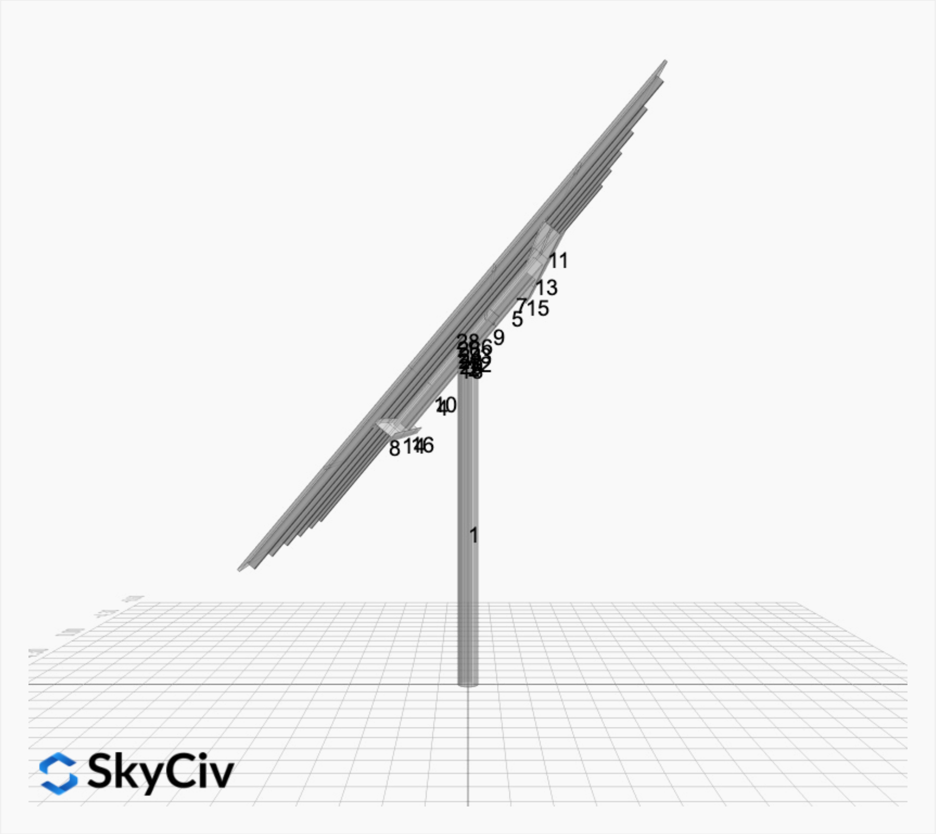
Design Notes:

- Deflection checks are set to L/1 due to manufacturer structural 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 7-16
- Steel frame design checks are based on AISC 360-16 LRFD
- Design / analysis of fixings and connections are not carried out by this module.
- Impacts of eccentrically applied, partial or pattern loading are not considered by this module.

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16
10
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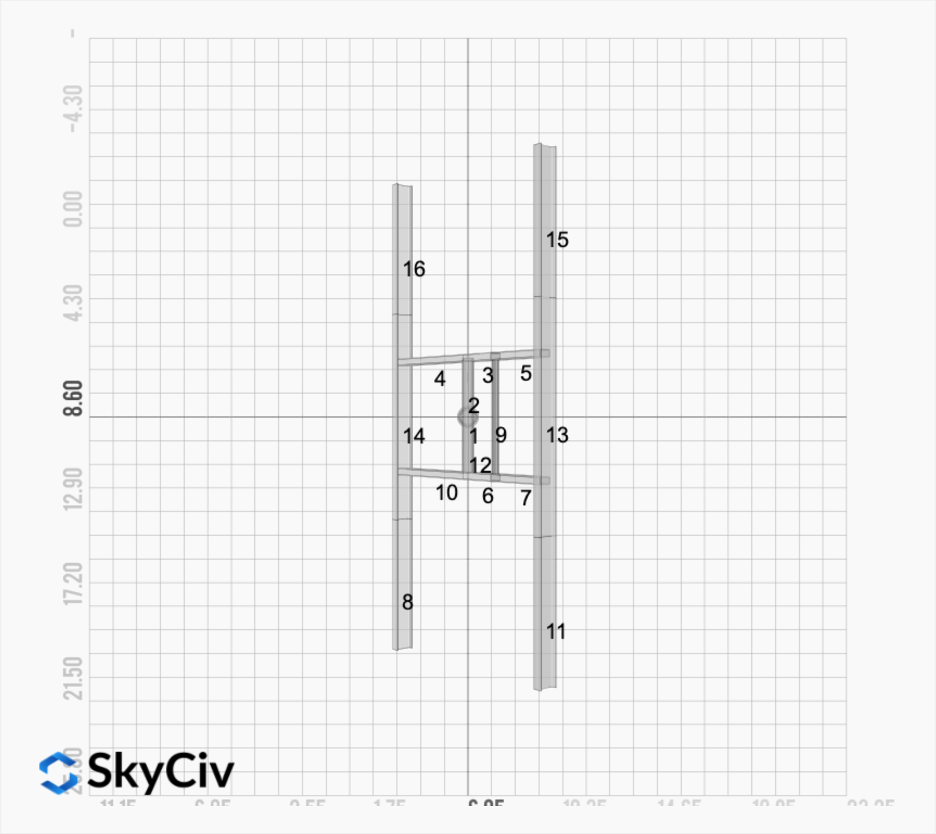
SkyCiv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0.00
4.30
8.60
12.90
17.20
21.50
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4
3
5
2
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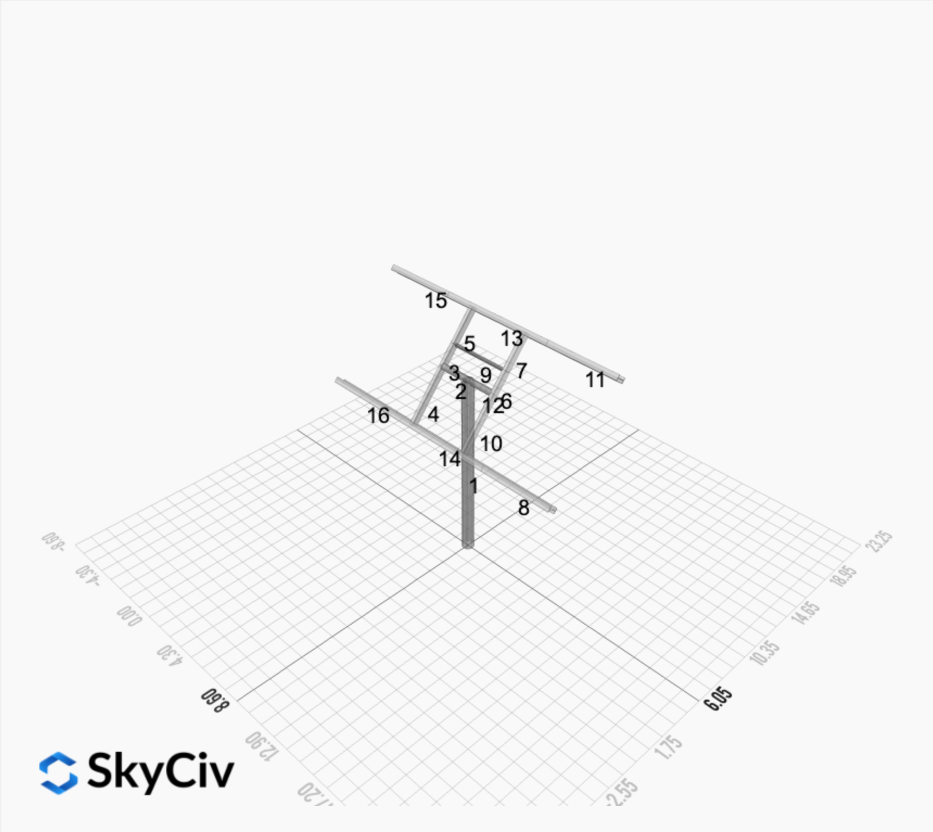
6
7
8
11
SkyCiv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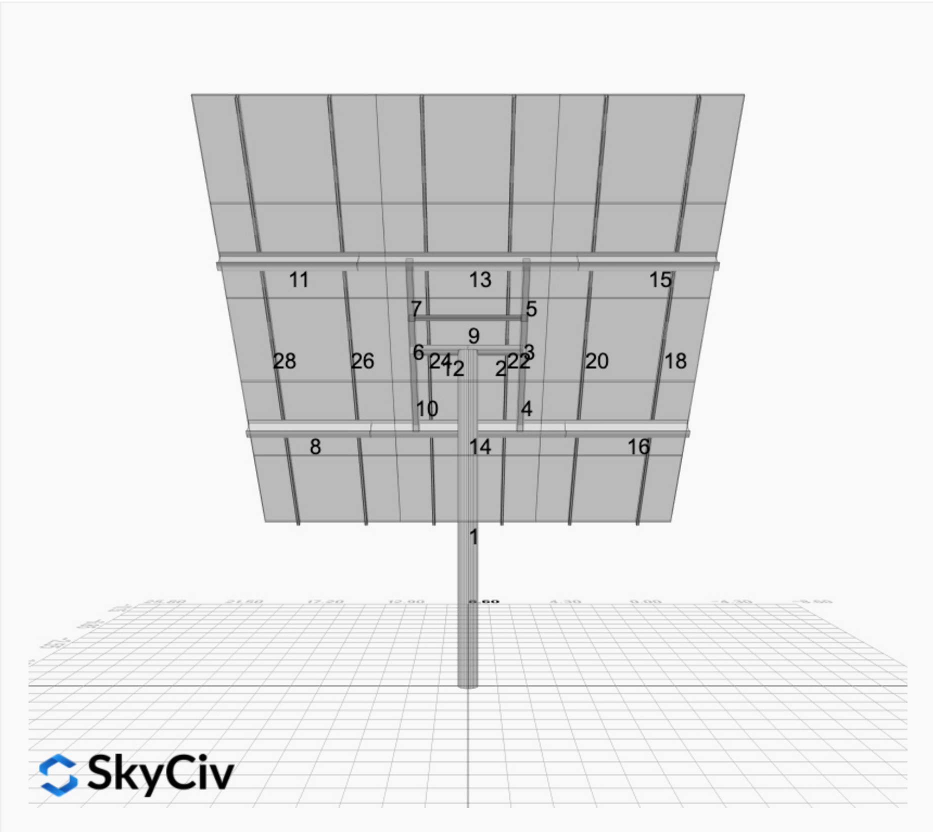
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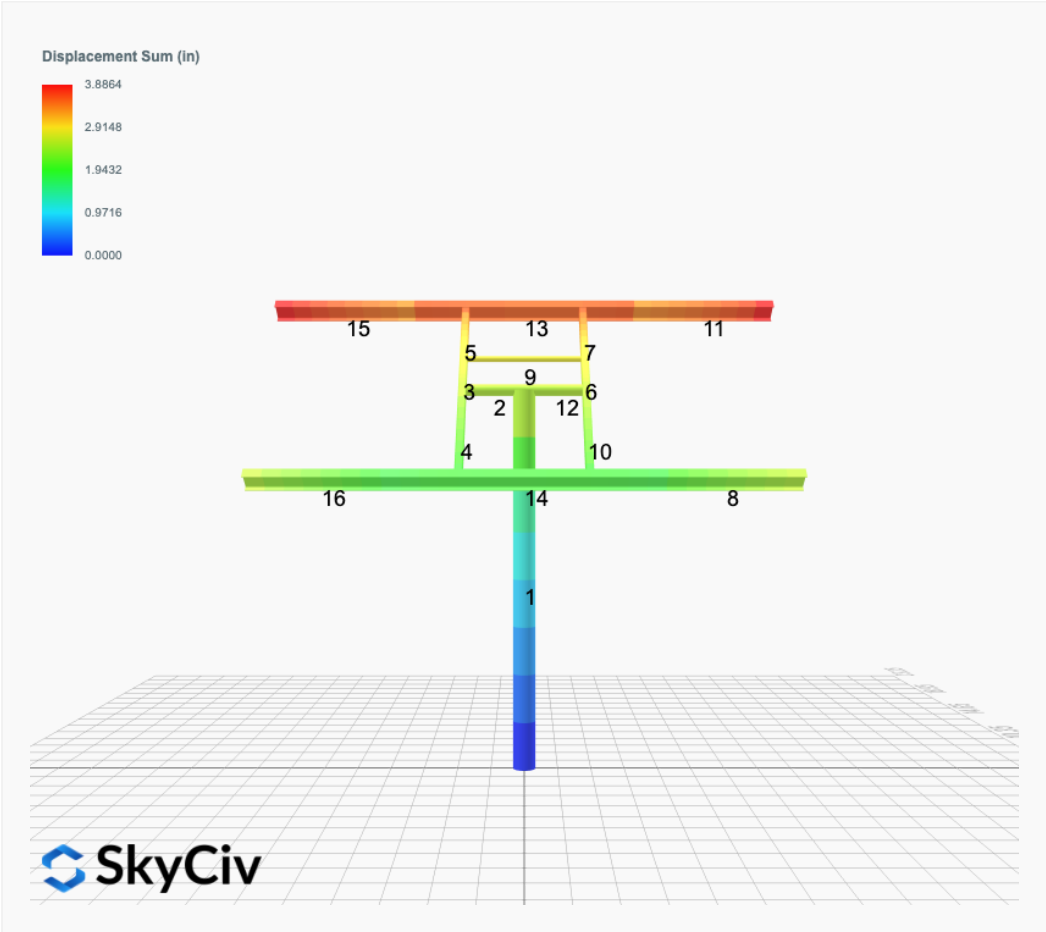
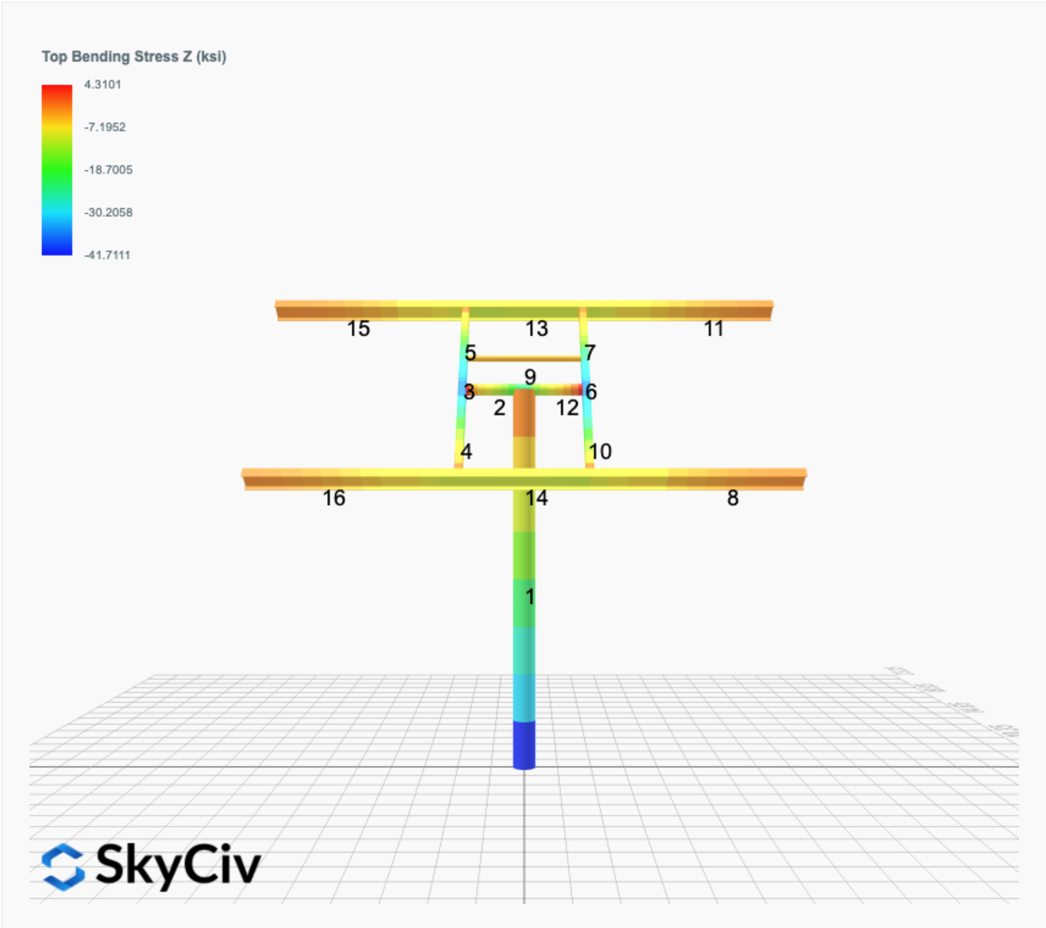
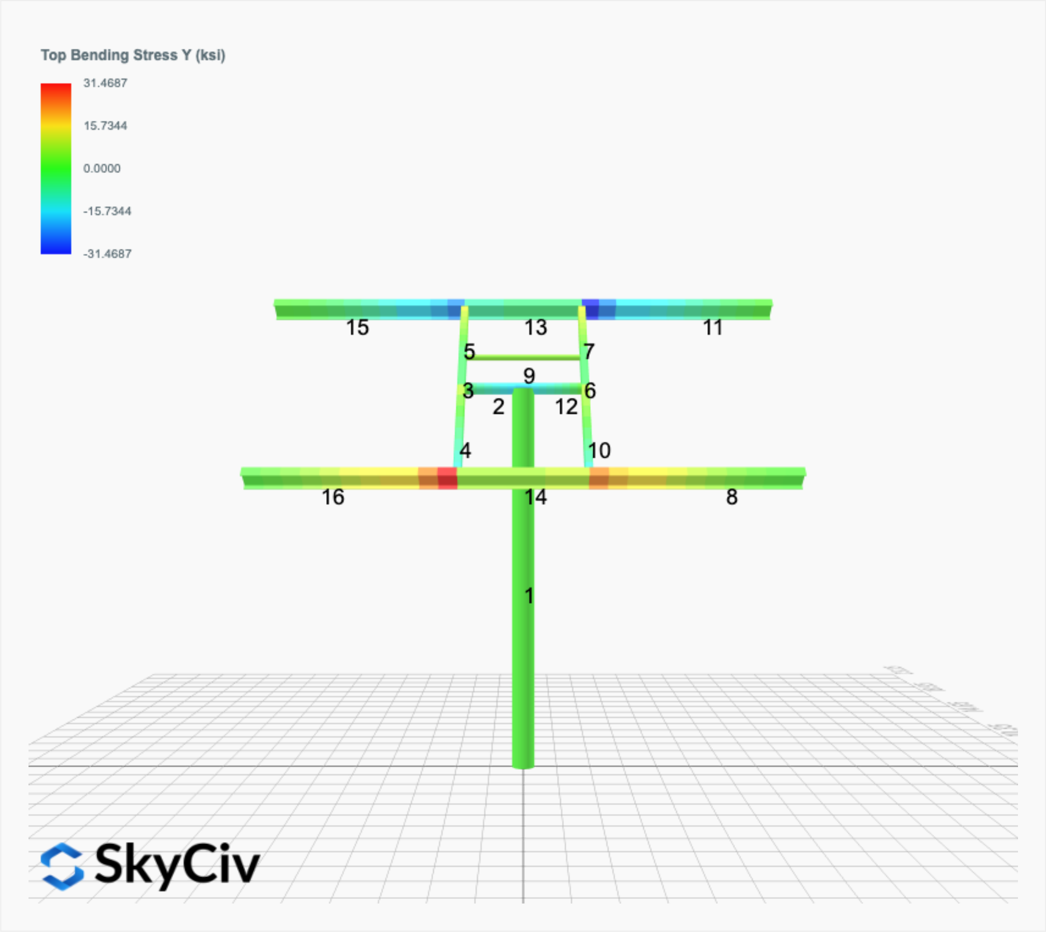
16
1
SkyCiv

FEM Results (Envelope Worst Case)



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0.0000
-15.7344
-31.4687
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5
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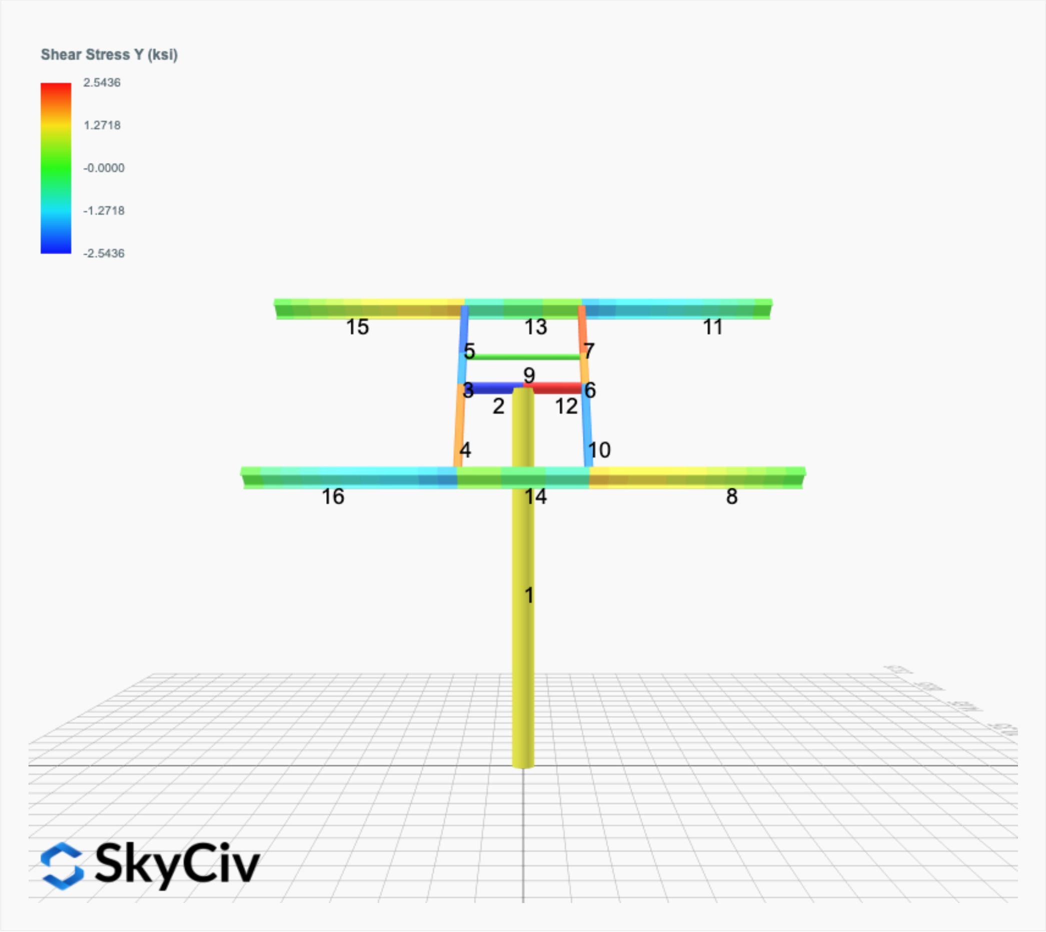
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
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-1.2718
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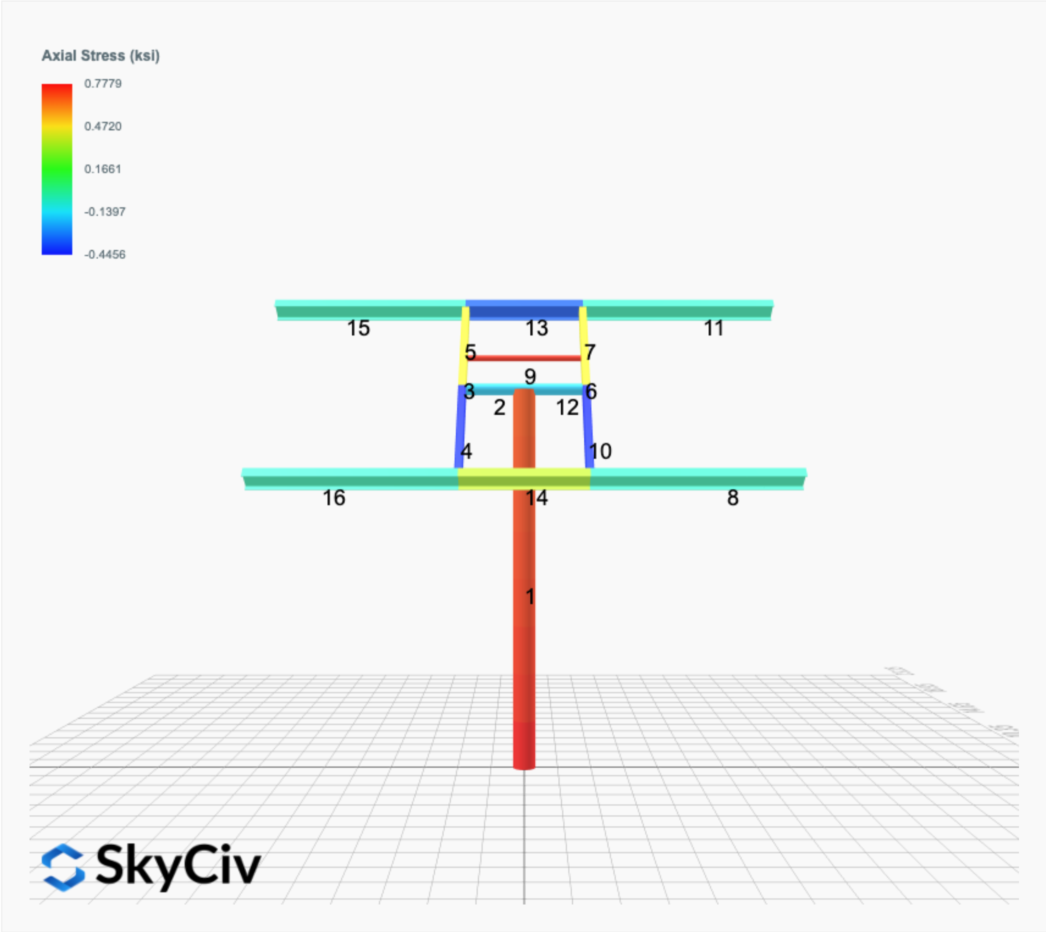
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
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13
11
5
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3
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2
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8
1
SkyCiv

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

LRFD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. 1.4D	0.0000	3.5846	0.0000	0.0000	-0.0000	0.0348
ULS: 2. 1.2D + 1.6L + 0.5(S or Lr or R)	0.0000	4.2173	0.0000	0.0000	-0.0000	0.0371
ULS: 2. 1.2D + 1.6L + 0.5(S or Lr or R)	0.0000	3.0725	0.0000	0.0000	-0.0000	0.0287
ULS: 3. 1.2D + 1.6(S or Lr or R) + L	0.0000	6.7359	0.0000	0.0000	-0.0000	0.0676
ULS: 5. 1.2D + E + L + 0.2S	0.0000	3.5304	0.0000	0.0000	-0.0000	0.0316
ULS: 7. 0.9D + 1.0E	0.0000	2.3044	0.0000	0.0000	-0.0000	0.0202
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind downforce Case A only	-6.8060	9.9282	0.0000	0.0000	0.0000	85.2085
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind downforce Case B only	0.0000	4.2173	0.0000	0.0000	-0.0000	0.0371
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind uplift Case A only	6.8060	-1.4936	0.0000	-0.0000	-0.0000	-82.8604
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind uplift Case B only	0.0000	4.2173	0.0000	0.0000	-0.0000	0.0371
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind downforce Case A only	-6.8060	8.7834	0.0000	0.0000	0.0000	84.9445
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind downforce Case B only	0.0000	3.0725	0.0000	0.0000	-0.0000	0.0287
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind uplift Case A only	6.8060	-2.6384	0.0000	-0.0000	-0.0000	-82.6287
ULS: 4. 1.2D + W + L + 0.5(S or Lr or R)_Wind uplift Case B only	0.0000	3.0725	0.0000	0.0000	-0.0000	0.0287
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind downforce Case A only	-3.4030	9.5914	0.0000	0.0000	0.0000	42.6422
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind downforce Case B only	0.0000	6.7359	0.0000	0.0000	-0.0000	0.0676
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind uplift Case A only	3.4030	3.8805	0.0000	-0.0000	-0.0000	-41.9299
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind uplift Case B only	0.0000	6.7359	0.0000	0.0000	-0.0000	0.0676
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind downforce Case A only	-3.4030	5.9279	0.0000	0.0000	0.0000	42.1974
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind downforce Case B only	0.0000	3.0725	0.0000	0.0000	-0.0000	0.0287
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind uplift Case A only	3.4030	0.2170	0.0000	-0.0000	-0.0000	-41.5755
ULS: 3. 1.2D + 1.6(S or Lr or R) + 0.5W_Wind uplift Case B only	0.0000	3.0725	0.0000	0.0000	-0.0000	0.0287
ULS: 6. 0.9D + 1.0W_Wind downforce Case A only	-6.8060	8.0153	0.0000	0.0000	0.0000	84.7690
ULS: 6. 0.9D + 1.0W_Wind downforce Case B only	0.0000	2.3044	0.0000	0.0000	-0.0000	0.0202
ULS: 6. 0.9D + 1.0W_Wind uplift Case A only	6.8060	-3.4065	0.0000	-0.0000	-0.0000	-82.4803
ULS: 6. 0.9D + 1.0W_Wind uplift Case B only	0.0000	2.3044	0.0000	0.0000	-0.0000	0.0202

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 2. D + L	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 3. D + (S or Lr or R)	0.0000	4.8501	0.0000	0.0000	-0.0000	0.0409
ULS: 3. D + (S or Lr or R)	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.2776	0.0000	0.0000	-0.0000	0.0351
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 5b. D + 0.7E	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.2776	0.0000	0.0000	-0.0000	0.0351
ULS: 8. 0.6D + 0.7E	0.0000	1.5362	0.0000	0.0000	-0.0000	0.0127
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.0836	5.9869	0.0000	0.0000	0.0000	50.6283
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.0836	-0.8661	0.0000	-0.0000	-0.0000	-49.7720
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0627	6.8476	0.0000	0.0000	0.0000	38.0812
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	4.2776	0.0000	0.0000	-0.0000	0.0351
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.0627	1.7077	0.0000	-0.0000	-0.0000	-37.5504
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	4.2776	0.0000	0.0000	-0.0000	0.0351

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.0627	5.1303	0.0000	0.0000	0.0000	37.8997
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.0627	-0.0095	0.0000	-0.0000	-0.0000	-37.3979
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.5604	0.0000	0.0000	-0.0000	0.0230
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.0836	4.9628	0.0000	0.0000	0.0000	50.4866
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	1.5362	0.0000	0.0000	-0.0000	0.0127
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.0836	-1.8903	0.0000	-0.0000	-0.0000	-49.6557
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	1.5362	0.0000	0.0000	-0.0000	0.0127

Worst Case Reactions (LRFD)

Note: Downforce / downwind wind load cases are assumed to govern.

Result	Value (kip, kip-ft)
Axial	9.9282
Shear X	-6.8060
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	85.2085

Worst Case Reactions (ASD)

Note: Downforce / downwind wind load cases are assumed to govern.

Result	Value (kip, kip-ft)
Axial	6.8476
Shear X	-4.0836
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	50.6283

Project Details

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

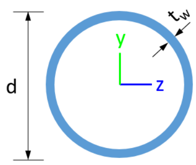
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

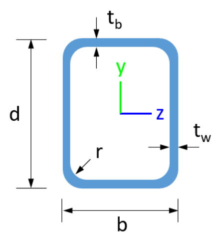
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65
2	29000	46	62
4	29000	50	62

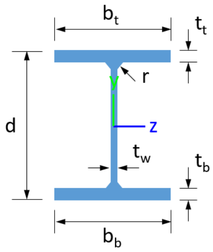
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
10	8in Pipe Sch 80	8.63	0.50					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17		



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties

Section Properties

ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties

Member ID	Section ID	K_z L (ft)	K_y L (ft)	L_b (ft)	C_b	LST	LSC	LD
1	10	7.94	7.94	12.21	-	300	200	250
2	5	1.30	1.30	2.00	-	300	200	250
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	250
4	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.63,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.68,1.68,1.67,1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	250
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	250
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	250
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68,1.68,1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	250
8	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	250
9	2	2.60	2.60	4.00	-	300	200	250
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.63,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.68,1.68,1.67,1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	250
11	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	250
12	5	1.30	1.30	2.00	-	300	200	250
13	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	250
14	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	250
15	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	250
16	19	9.97	9.97	4.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	250

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	528.38	490.83	114.02	114.02	158.51	158.51
2	182.47	181.10	20.19	20.19	54.74	54.74

3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	32.95	32.87	6.12	40.24	43.62
9	61.16	54.71	3.51	3.51	18.35	18.35
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	32.95	32.87	6.12	40.24	43.62
12	182.47	181.10	20.19	20.19	54.74	54.74
13	133.20	85.85	23.60	6.12	40.24	43.62
14	133.20	85.85	23.59	6.12	40.24	43.62
15	133.20	32.95	32.87	6.12	40.24	43.62
16	133.20	32.95	32.87	6.12	40.24	43.62

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.020	0.747	0.000	0.043	0.000	0.757	#13	0.166	Not Required	Pass
2	0.005	0.382	0.331	0.085	0.062	0.715	#13	0.035	Not Required	Pass
3	0.010	0.663	0.050	0.066	0.002	0.696	#13	0.045	Not Required	Pass
4	0.010	0.660	0.190	0.066	0.039	0.732	#13	0.080	Not Required	Pass
5	0.010	0.412	0.197	0.066	0.050	0.442	#13	0.074	Not Required	Pass
6	0.010	0.663	0.050	0.066	0.002	0.696	#13	0.045	Not Required	Pass
7	0.010	0.412	0.197	0.066	0.050	0.442	#13	0.074	Not Required	Pass
8	0.000	0.111	0.243	0.038	0.014	0.326	#21	Not Required	Not Required	Pass
9	0.021	0.050	0.069	0.001	0.000	0.123	#13	0.204	Not Required	Pass
10	0.010	0.660	0.190	0.066	0.039	0.732	#13	0.080	Not Required	Pass
11	0.000	0.111	0.243	0.038	0.014	0.326	#21	Not Required	Not Required	Pass
12	0.005	0.382	0.331	0.085	0.062	0.715	#13	0.035	Not Required	Pass
13	0.009	0.304	0.455	0.052	0.020	0.670	#21	0.190	Not Required	Pass
14	0.011	0.309	0.455	0.052	0.020	0.671	#21	0.190	Not Required	Pass
15	0.000	0.111	0.243	0.038	0.014	0.326	#21	Not Required	Not Required	Pass
16	0.000	0.111	0.243	0.038	0.014	0.326	#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/comression)
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

IBC 2018 Pile Design

Powered by SkyCiv

Input	Description
Region	American Standard
Concrete design code	American Concrete Institute (ACI 318:2019)

Cross-section

Input	Description	Value
Shape	Cross-sectional shape	Square
b	Section width	84 in
D	Section deph	84 in

Material Properties

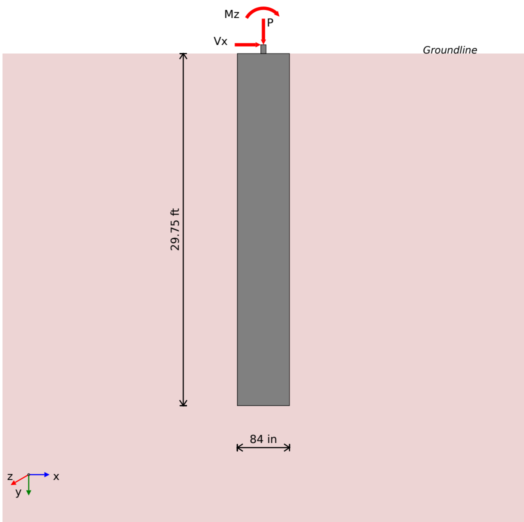
Input	Description	Value
f'_{ck}	Concrete compressive strength	2.5 ksi
f_{yk}	Yield strength of steel	60 ksi
d_b	Rebar diameter	#5 (0.625) in
cover	Concrete cover	3 in

Soil Parameters (IBC 1806)

Input	Description	Value
Soil type	Sand, silty sand, clayey sand, silty gravel & clayey gravel	
q_a	Allowable bearing pressure	2000 psf
R	Allowable lateral pressure	150 psf/ft

Loading

Load	ASD	LRFD
P	6.848 kip	9.928 kip
Vx	-4.084 kip	-6.806 kip
Vz	0 kip	0 kip
Mx	0 kip-ft	0 kip-ft
Mz	50.63 kip-ft	85.21 kip-ft



REFERENCES	CALCULATIONS	RESULTS

Required depth to resist lateral loads (ASD)

Allowable lateral pressure $R =$ 150 psf/ft

Point of application of lateral load:

$$H = h_1 + h_2 + h_e = 0 + 0 + 0 = 0\ ft$$

Considering x-direction:

Lateral force per section length

$$H_o = \frac{V_x}{1.57 \times D} = \frac{-4.084}{1.57 \times 84} = -0.372\ \frac{kip}{ft}$$

Moment per section length

$$M_o = \frac{M_z + (V_x \times H)}{1.57 \times D} = \frac{50.63 + (-4.084 \times 0)}{1.57 \times 84} = 4.607\ \frac{kip - ft}{ft}$$

Required depth of embedment in earth:

$$L_x^3 - \left(9 \times \frac{H_o \times L_x}{R}\right) - \left(12 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:

$$L_{e,x} = 6.141\ ft$$

Considering z-direction:

Since there are no loads applied in this direction, the required effective length: $L_{e,z}$ = 0 ft.

Minimum embedded depth

Depth of pile required

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}] = MAX[6.141, 0] = 6.141\ ft$$

Actual embedded length

$$L_e = L - h_2 - h_e = 29.75 - 0 - 0 = 29.75\ ft$$

Utilisation

$$Ratio = \frac{L_{e,req}}{L_e} = \frac{6.141}{29.75} = 0.206$$

UTILITY: 0.21

REFERENCES	CALCULATIONS	RESULTS

End-bearing Capacity (ASD)

Allowable bearing pressure	$q_a =$	2000 psf
Unit weight of concrete	$w_c =$	0.15 kip/ft^3

Cross-sectional area:

$$A = b \times D = 84 \times 84 = 49\ ft^2$$

End-bearing pressure:

$$q = \frac{P}{A} = \frac{6.848}{49} = 139.7\ psf$$

Utilisation

$$Ratio = \frac{q}{q_a} = \frac{139.7}{2000} = 0.07$$

UTILITY: 0.07

Lateral Soil Pressure (ASD)

Allowable lateral pressure	$R =$	150 psf/ft

Length to least lateral dimension ratio:

$$\frac{L}{MIN[b, D]} = \frac{29.75}{MIN[7, 7]} = 4.25$$

L/D ratio ≤ 10. This pile is classified as a short pile.

Considering x-direction:

Distance from resting surface to pivot point:

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

$$a = \frac{(4 \times 4.607 \times 29.75) + \left(3 \times 0.372 \times 29.75^2\right)}{(6 \times 4.607) + (4 \times 0.372 \times 29.75)} = 21.36\ ft$$

Earth pressure against the pile at a distance a/2 from the resting surface:

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

$$p = \frac{0.75 \times [(4 \times 4.607) + (3 \times -0.372 \times 29.75)]^2}{29.75^2 \times [(3 \times 4.607) + (2 \times -0.372 \times 29.75)]} = -0.022\ \frac{kip}{ft^2}$$

Allowable lateral soil pressure at a depth of a/2:

$$p_a = R \times \frac{a}{2} = 0.15 \times \frac{21.36}{2} = 1.602 \, \frac{kip}{ft^2}$$

Utilisation - pressure at a depth of a/2

$$Ratio = \frac{p}{p_a} = \frac{-0.022}{1.602} = -0.014$$

UTILITY: 0.01

Earth pressure against the pile at distance L_e:

$$s = \frac{6 \times [(2 \times M_o) + (H_o \times L_e)]}{L_e^2} = \frac{6 \times [(2 \times 4.607) + (-0.372 \times 29.75)]}{29.75^2} = -0.012 \, \frac{kip}{ft^2}$$

Allowable lateral soil pressure at a depth of L_e:

$$p_s = R \times L_e = 0.15 \times 29.75 = 4.462 \, \frac{kip}{ft^2}$$

Utilisation - pressure at a depth of L_e

$$Ratio = \frac{s}{p_s} = \frac{-0.012}{4.462} = -0.003$$

UTILITY: 0.00

Considering z-direction:

Since no loads are applied in this direction, lateral soil pressure check is not required.

REFERENCES	CALCULATIONS	RESULTS

Shear force and bending moment (LRFD)

Considering x-direction:

Lateral force per section length

$$H_o = \frac{V_x}{1.57 \times D} = \frac{-6.806}{1.57 \times 84} = -0.619 \, \frac{kip}{ft}$$

Moment per section length

$$M_o = \frac{M_z + (V_x \times H)}{1.57 \times D} = \frac{85.21 + (-6.806 \times 0)}{1.57 \times 84} = 7.753 \, \frac{kip - ft}{ft}$$

Distance from resting surface to pivot point:

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

$$a = \frac{(4 \times 7.753 \times 29.75) + \left(3 \times 0.619 \times 29.75^2\right)}{(6 \times 7.753) + (4 \times 0.619 \times 29.75)} = 21.35 \, ft$$

Max shear force located at depth a:

$$E = \frac{M_o}{H_o} = \frac{7.753}{-0.619} = 12.52 \, ft$$

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

$$V_{max,x} = (-0.619 \times 84) \times [1 - [3 \times \left(\frac{4 \times 12.52}{29.75} + 3\right) \times \left(\frac{21.35}{29.75}\right)^2] + [4 \times \left(\frac{3 \times 12.52}{29.75} + 2\right) \times \left(\frac{21.35}{29.75}\right)^3]]$$

$$V_{max,x} = 6.124 \, kip$$

Max bending moment located at a depth of a/2:

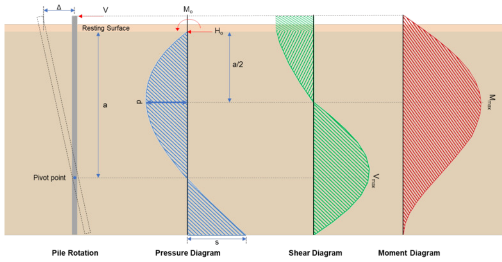
$$M_{max,x} = (H_o \times D \times L_e) \times [\left(\frac{E}{L_e} + \frac{a}{2 \times L_e}\right) - [\left(\frac{4 \times E}{L_e} + 3\right) \times \left(\frac{a}{2 \times L_e}\right)^3] + [\left(\frac{3 \times E}{L_e} + 2\right) \times \left(\frac{a}{2 \times L_e}\right)^4]]$$

$$M_{max,x} = (-0.619 \times 84 \times 29.75) \times [\left(\frac{12.52}{29.75} + \frac{21.35}{2 \times 29.75}\right) - [\left(\frac{4 \times 12.52}{29.75} + 3\right) \times \left(\frac{21.35}{2 \times 29.75}\right)^3] + [\left(\frac{3 \times 12.52}{29.75} + 2\right) \times \left(\frac{21.35}{2 \times 29.75}\right)^4]]$$

$$M_{max,x} = 79.62 \, kip - ft$$

Considering z-direction:

There are no loads applied in this direction.



Minimum Reinforcement Check (LRFD)

Gross area of concrete:

$$A_g = b \times D = 84 \times 84 = 7056\, in^2$$

Main Reinforcement

22.4.2.2 Required reinforcement:

$$A_{st,req} = \frac{P - (0.85 \times f'_{ck} \times A_g)}{f_{yk} - (0.85 \times f'_{ck})} = \frac{9.928 - (0.85 \times 2.5 \times 7056)}{60 - (0.85 \times 2.5)} = -258.9\, in^2$$

10.6.1.1 Maximum reinforcement:

$$A_{st,max} = 0.08 \times A_g = 0.08 \times 7056 = 564.5 in^2$$

7.6.1.1 Minimum reinforcement:

$$A_{st,min} = 0.0018 \times A_g = 0.0018 \times 7056 = 12.7 in^2$$

Governing minimum reinforcement area:

$$(0.0018 \times A_g) \leq A_{st,req} \leq (0.08 \times A_g)$$

$$A_{min} = 12.7\, in^2$$

Minimum number of reinforcements:

$$A_{bar} = 0.307\, in^2$$

$$n_{min} = \frac{A_{min}}{A_{bar}} = \frac{12.7}{0.307} = 42$$

25.2.3 Minimum spacing:

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

$$\text{Use: } n = 44\text{pcs at } 1.5 \text{ in minimum spacing}$$

Total reinforcement area:

$$A_{st} = 44 \times 0.307 = 13.5\, in^2$$

Shear Reinforcement

25.7.2.2 For main reinforcement $\lesssim$ 1.41 in: Use #3(0.375 in)

Maximum spacing of shear Reinforcements:

$$s = MIN[16 \times d_b, 48 \times d_{b,ties}, MIN\,(b, D)] = MIN[(16 \times 0.625)\,, (48 \times 0.375)\,, MIN\,(84, 84)] = 10\, in$$

Detailing Summary	
Main reinforcement	#5 (0.625 in) - 44pcs at 1.5 in min. spacing
Shear reinforcement	#3 (0.375 in) at 10 in max. spacing

Axial Compression Strength (LRFD)

22.4.2.2 Allowable axial compressive strength:

$$\phi P_N = \phi \times 0.8 \times [(0.85 \times f'_{ck} \times [A_g - A_{st}]) + (f_{yk} \times A_{st})]$$

$$\phi P_N = 0.65 \times 0.8 \times [(0.85 \times 2.5 \times [7056 - 13.5]) + (60 \times 13.5)] = 8203\, kip$$

Utilisation

$$Ratio = \frac{P}{\phi P_N} = \frac{9.928}{8203} = 0.001$$

ϕP_N 8203

UTILITY: 0.00

Shear Strength LRFD)

Effective shear width	$b_w =$	84 in
Effective shear depth	$d =$	80.31 in
Shear reinforcement area	$A_v =$	0.221 in^2
Shear reinforcement spacing	$s =$	10 in
Concrete type factor (Normal concrete)	$\lambda =$	1
Strength reduction factor for shear	$\phi =$	0.75
Maximum shear in the x-direction	$V_{max,x} =$	6.124 kip
Maximum shear in the z-direction	$V_{max,z} =$	0 kip

22.5.5.1.1 Max shear strength of concrete:

$$V_{c,max} = 5 \times \lambda \times \sqrt{f'_{ck}} \times b_w \times d = 5 \times 1 \times \sqrt{2.5} \times 84 \times 80.31 = 1687\, kip$$

Table 22.5.5.1 Shear strength of concrete:

$$V_{c,a} = \left(2 \times \lambda \times \sqrt{f'_{ck}} + MIN[\frac{P}{6 \times A_g}, (0.05 \times f'_{ck})]\right) \times (b_w \times d)$$

$$V_{c,a} = \left(2 \times 1 \times \sqrt{2.5} + MIN[\frac{9.928}{6 \times 7056}, (0.05 \times 2.5)]\right) \times (84 \times 80.31) = 676.2\, kip$$

Governing shear strength of concrete:

$$V_c = MIN[V_{c,max}, V_{c,a}] = MIN[1687, 676.2] = 676.2\, kip$$

22.5.1.2 Shear strength of steel (a):

$$V_{s,a} = 8 \times \sqrt{f'_{ck}} \times b_w \times d = 8 \times \sqrt{2.5} \times 84 \times 80.31 = 2699\, kip$$

22.5.8.5.3 Shear strength of steel (b):

$$V_{s,b} = \frac{A_v \times f_{yk} \times d}{s} = \frac{0.221 \times 60 \times 80.31}{10} = 106.4\, kip$$

Governing shear strength of steel:

$$V_s = MIN[V_{s,a}, V_{s,b}] = MIN[2699, 106.4] = 106.4\, kip$$

22.5.1.1 Allowable shear strength:

$$\phi V_n = \phi \times (V_c + V_s) = 0.75 \times (676.2 + 106.4) = 587\, kip$$

$$V_{max} = MAX[6.124, 0] = 6.124\, kip$$

Utilisation

$$Ratio = \frac{V_{max}}{\phi V_n} = \frac{6.124}{587} = 0.01$$

UTILITY: 0.01

Flexural Strength (LRFD)

Concrete type factor (Normal concrete)	$\lambda =$	1
Strength reduction factor for flexure	$\phi =$	0.65
Modulus of steel reinforcement	$E_s =$	200e3 ksi
Maximum concrete strain	$\varepsilon_c =$	0.0030
Yield strain of steel f_y/E_s	$\varepsilon_y =$	0.0003
Section width	$b =$	84 in
Distance to the compression rebar	$d_s =$	3.688 in
Distance to the tension rebar	$d =$	80.31 in
Total bar area	$A_s =$	13.5 in^2
Maximum applied axial load	$P =$	9.928 kip
Maximum moment in the x-direction	$M_{max,x} =$	79.62 kip-ft
Maximum moment in the z-direction	$M_{max,z} =$	0 kip-ft

Compressive force due to concrete:

$$\beta_1 = 0.85$$

$$C_{rc} = 0.85 \times \beta_1 \times f'_c \times b \times c$$

Compressive force due to bars in compression:

$$C_{rs} = f_1 \times A_{sc}$$

$$\varepsilon_1 = (c - d_s) \times \frac{\varepsilon_c}{c}$$

$$f_1 = E_s \times \varepsilon_1 \ (\varepsilon_1 < \varepsilon_{sy}),\ f_1 = f_y \ (\varepsilon_1 \geq \varepsilon_{sy})$$

Tensile force due to bars in tension:

$$T_{rs} = f_2 \times A_{st}$$

$$\varepsilon_2 = (d - c) \times \frac{\varepsilon_{cu}}{c}$$

$$f_2 = E_s \times \varepsilon_2 \ (\varepsilon_2 < \varepsilon_{sy}),\ f_2 = \phi_s \times f_y \ (\varepsilon_2 \geq \varepsilon_{sy})$$

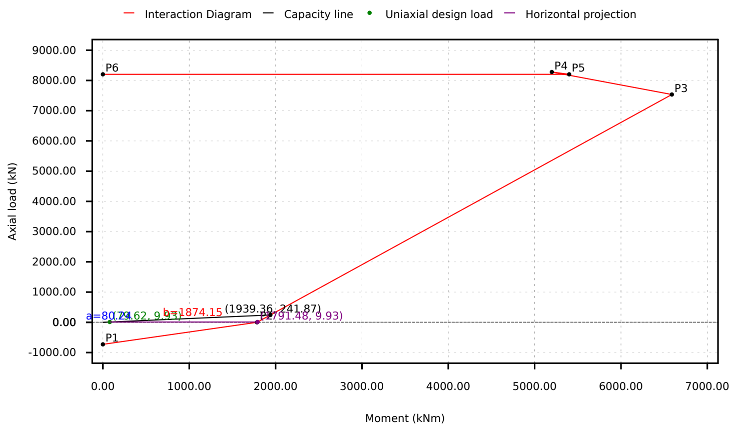
Interaction Diagram Summary

Point	Case	M_r	P_r
P1	Pure Tension	0	-728.9
P2	Pure Bending	1785	0
P3	Balanced Failure	6589	7535
P4	Decompression	5199	8279
P5	Compression Limit	5400	8203
P6	Pure Compression	0	8203

Uniaxial Bending Check

$$M_f = MAX[79.62, 0] = 79.62\ kip - ft$$

Interaction Diagram



Segment	Signed Distance
P1 - P2	653.9
P2 - P3	1443
P3 - P4	9707
P4 - P5	-9548
P5 - P6	8193
Status	FAIL: Point lies outside the curv e

Utilisation

$$Ratio = \frac{a + b}{a} = \frac{80.24 + 1874}{80.24} = 24.36$$

UTILITY: 24.36

REFERENCES	CALCULATIONS	RESULTS

Results Summary

Result Name	Results
PILE DETAILS	
Length of the pile	29.75 ft
Dimensions	84 x 84 in
Main bar reinforcement	#5-44pcs at 1.5 in min.
Shear reinforcement	#3 at 10 in max.
UTILISATIONS	
Required depth	0.21
End-bearing capacity	0.07
p_a	0.00
p_s	0.00
Axial compression strength	0.00
Shear strength	0.01
Uniaxial bending strength	24.36