Your Project Calculations

Project Name: Dushane-RevA

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Dushane-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/2_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=RFieHx0Sy2wvvxstlHd5BVVYh2Wi551quSJbpje0nqiAs5niULzdn0ftzYz2aCF7

Array Specification

Product:	Beam
Unique ID:	2P-17-6TOP-HD-24-L-4Hx4W-B75B
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	30
Front Edge Clearance:	7
Total Array Height at Tilt:	13.89 ft
Total Frame Length:	28.50 ft
Frame Weight:	1395 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	29.40 ft
Rail Length:	166.40 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.47 ft
Number of Poles:	2
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.25 ft
Foundation Volume:	6.222 y^3
Foundation Result:	PASSED

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Forestdale Cemetery, 2022 Creek Rd, Crown Point, NY 12928, USA
Wind Speed:	100 mph
Snow Load:	50 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.019046 ksf

Design Disclaimer

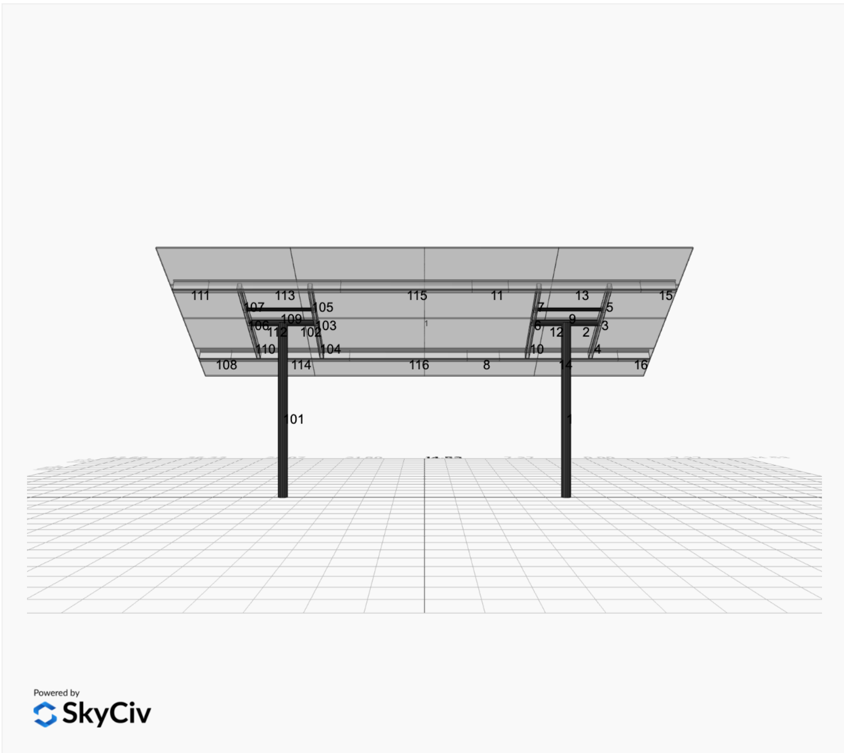
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

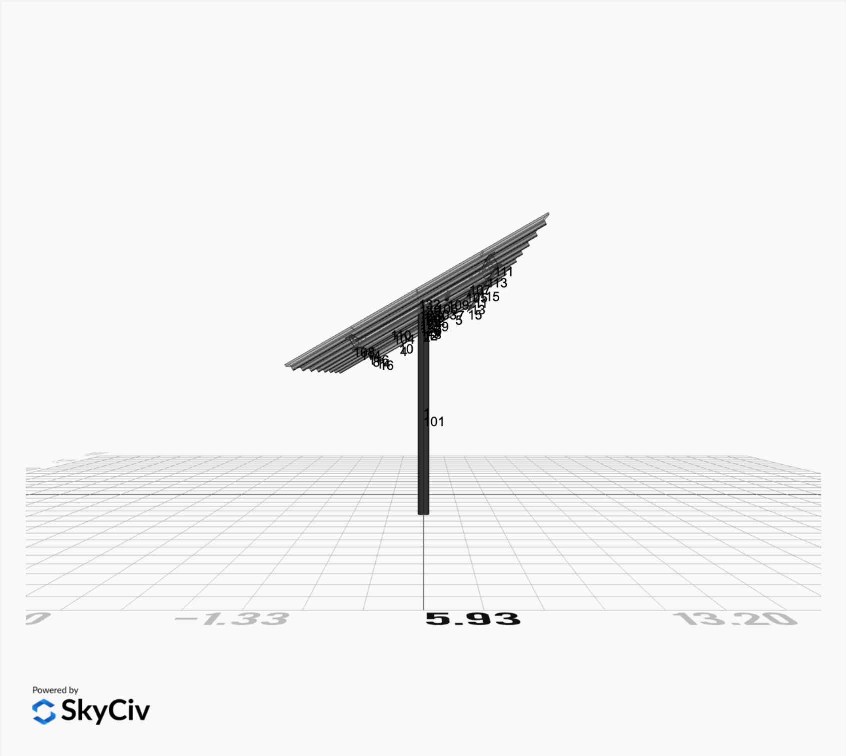
AutoDesigner Input

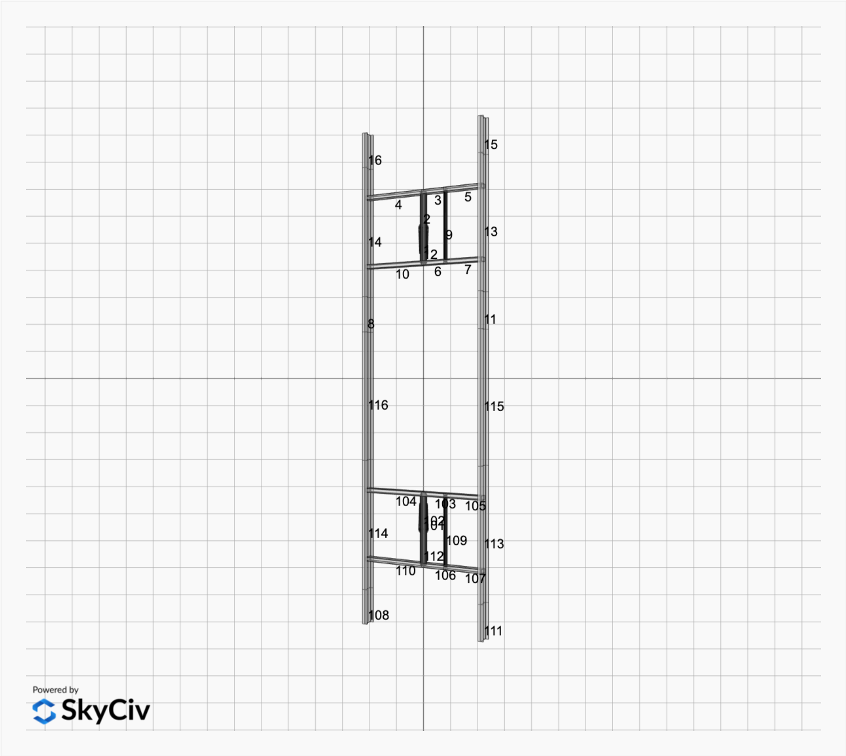
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"product_type":"Beam","project_id":"Dushane-RevA","site_address":"Forestdale Cemetery, 2022 Creek Rd, Crown Point, NY 12928, USA","module_width":41.1,"module_length":87.2,"number_rows":"4","number_columns":4,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":30,"ground_clearance":7,"risk_category":"I","exposure_category":"B","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","foundation_type":"Square","foundation_size":48,"check_rails":false}

Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only




-1.33
5.93
13.20
Powered by
SkyCiv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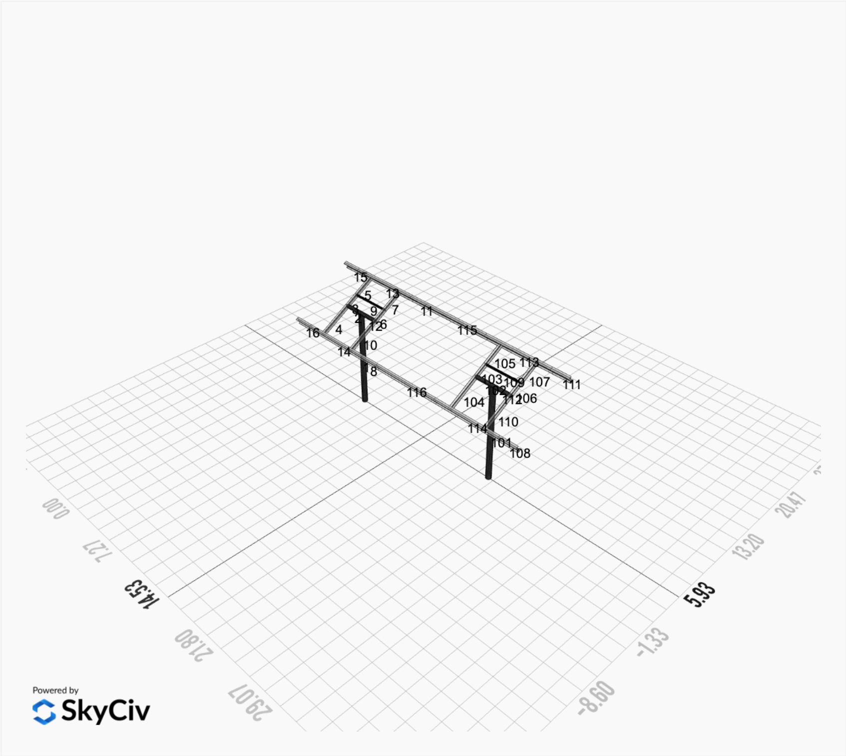
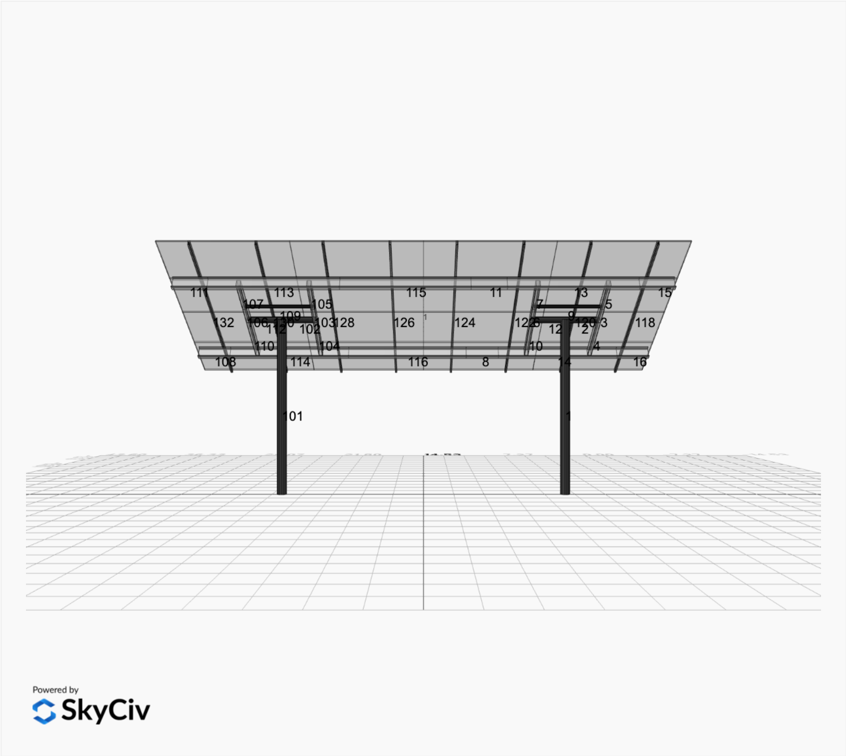
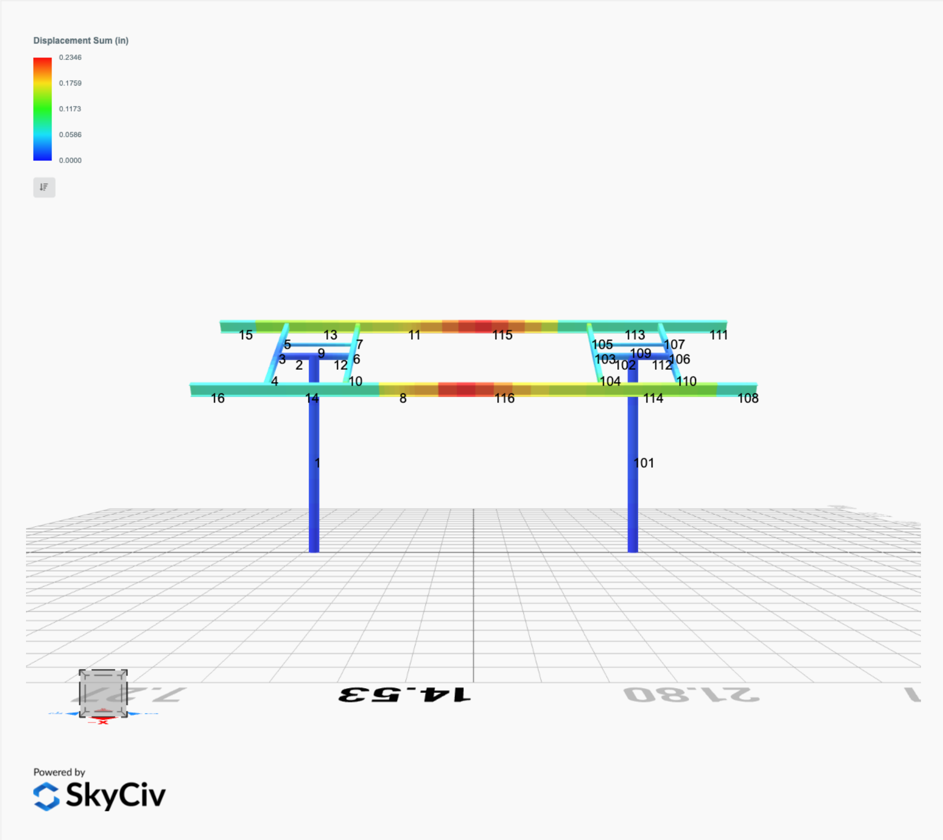
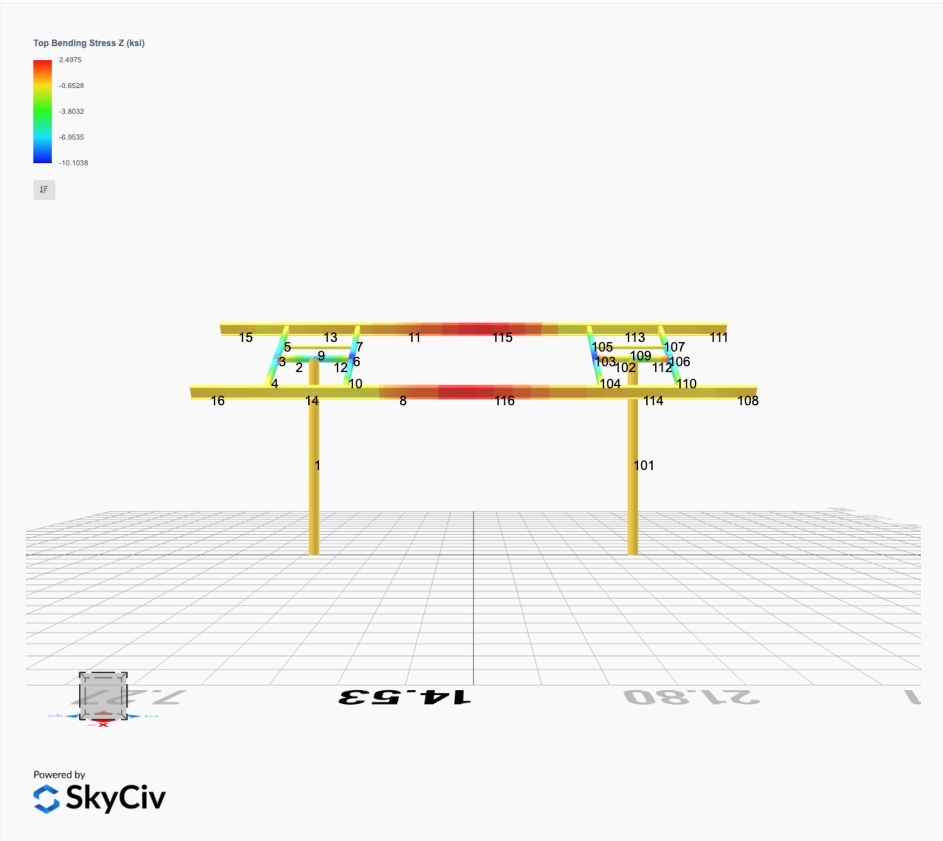
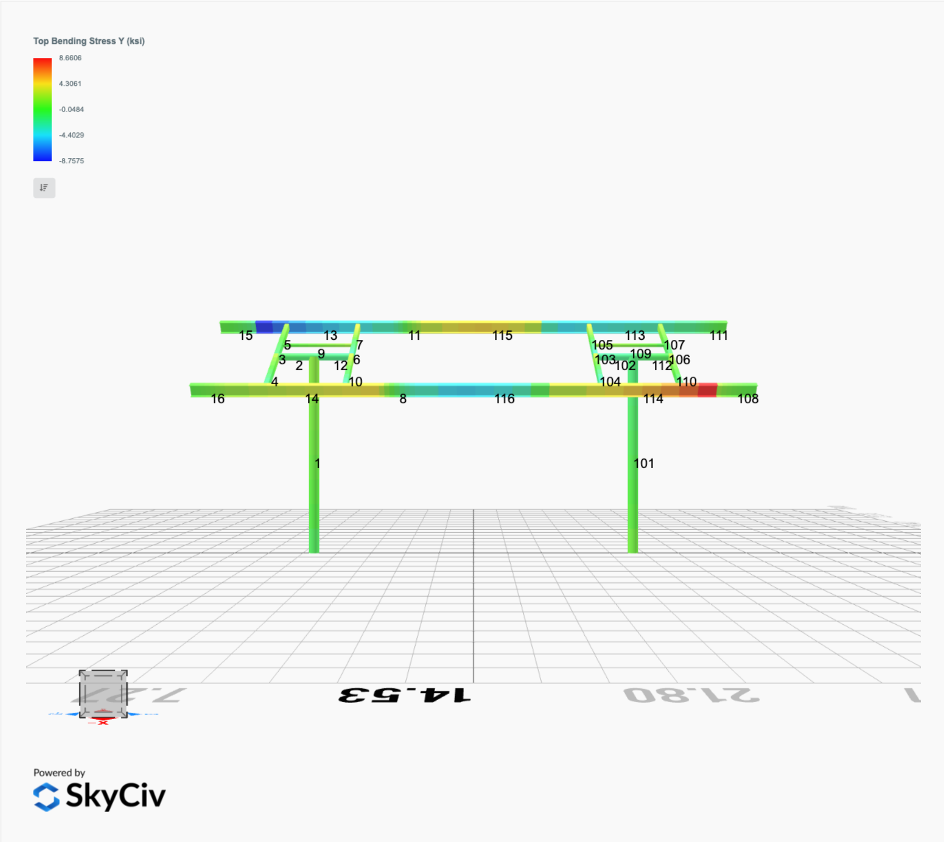


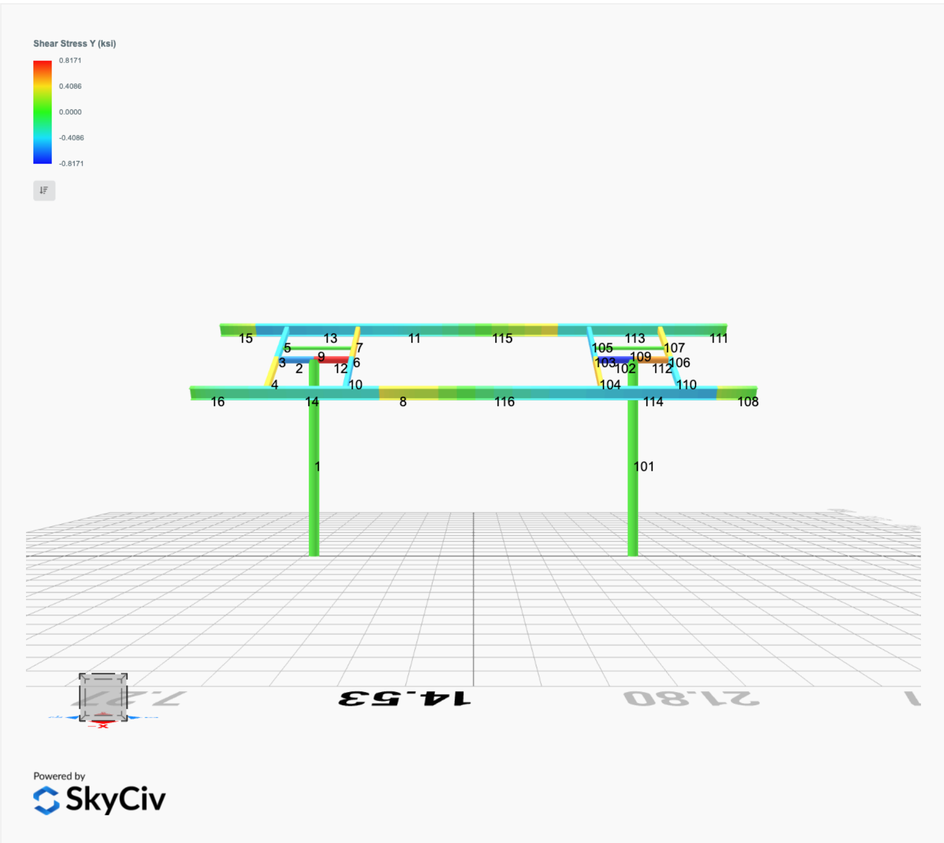


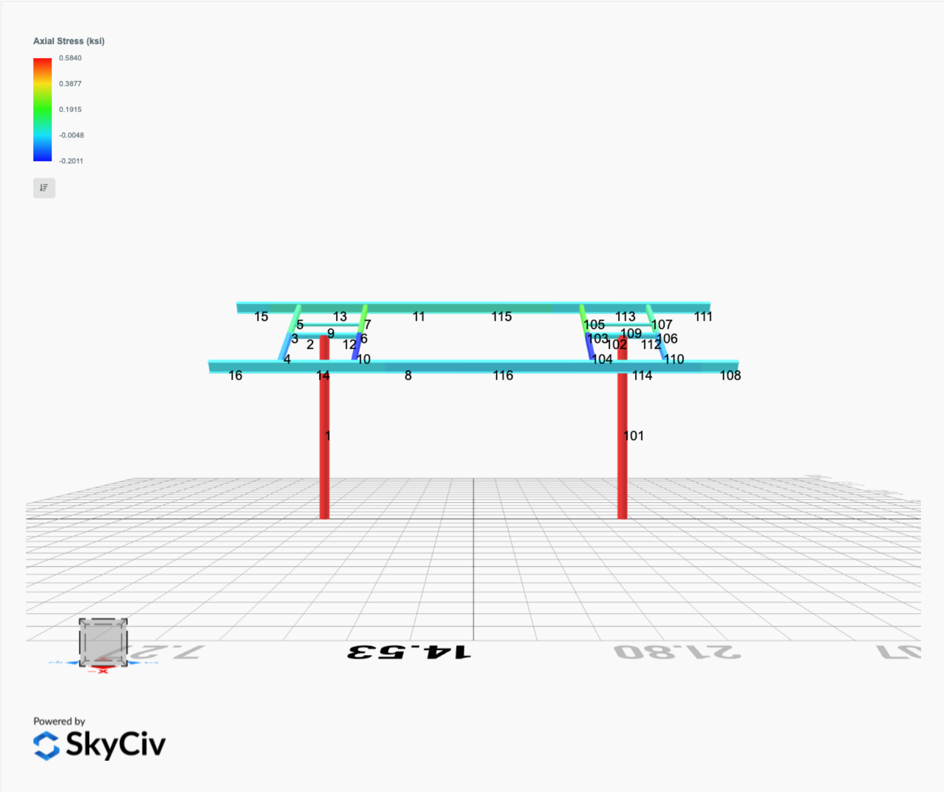
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
8.6606
4.3061
-0.0484
-4.4029
-8.7575
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
Powered by
SkyCiv


Shear Stress Y (ksi)
0.8171
0.4086
0.0000
-0.4086
-0.8171
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
Powered by
SkyCiv


Axial Stress (ksi)
0.5840
0.3877
0.1915
-0.0048
-0.2011
15
13
11
115
113
111
5
7
105
107
9
109
3
2
12
6
103
102
112
106
4
10
104
110
16
14
8
116
114
108
1
101
14.53
21.80
Powered by
SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0000	1.6330	0.0368	0.1216	-0.0202	0.0260
ULS: 2. D + L	0.0000	1.6330	0.0368	0.1216	-0.0202	0.0260
ULS: 3. D + (S or Lr or R)	0.0000	4.8923	0.1356	0.4484	-0.0748	0.0383
ULS: 3. D + (S or Lr or R)	0.0000	1.6330	0.0368	0.1216	-0.0202	0.0260
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	4.0775	0.1109	0.3667	-0.0611	0.0353
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.6330	0.0368	0.1216	-0.0202	0.0260
ULS: 5b. D + 0.7E	0.0000	1.6330	0.0368	0.1216	-0.0202	0.0260
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	4.0775	0.1109	0.3667	-0.0611	0.0353
ULS: 8. 0.6D + 0.7E	0.0000	0.9798	0.0221	0.0730	-0.0121	0.0156
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3563	3.9822	0.1188	0.3872	-0.1420	14.5917
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3563	3.9822	0.1188	0.3872	-0.1420	14.5917
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1626	-0.3807	-0.0333	-0.1046	0.0842	-11.8858
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9688	-0.0450	-0.0221	-0.0686	0.0675	-14.6515
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0173	5.8394	0.1724	0.5659	-0.1525	10.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0173	5.8394	0.1724	0.5659	-0.1525	10.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8719	2.5672	0.0583	0.1971	0.0171	-8.8986
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7266	2.8190	0.0667	0.2241	0.0047	-10.9728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0173	3.3949	0.0983	0.3208	-0.1115	10.9503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0173	3.3949	0.0983	0.3208	-0.1115	10.9503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8719	0.1228	-0.0158	-0.0480	0.0581	-8.9079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7266	0.3745	-0.0074	-0.0211	0.0456	-10.9821
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3563	3.3290	0.1041	0.3385	-0.1339	14.5813
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3563	3.3290	0.1041	0.3385	-0.1339	14.5813
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1626	-1.0338	-0.0480	-0.1532	0.0922	-11.8962
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9688	-0.6982	-0.0369	-0.1173	0.0756	-14.6618

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value
Axial	9.1322
Shear X	-2.2606
Shear Z	0.2707
Moment X	0.8940
Moment Z	25.0956

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
Axial	5.8394
Shear X	-1.3563
Shear Z	0.1724
Moment X	0.5659
Moment Z	14.6618

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0000	1.6330	-0.0368	-0.1216	0.0202	0.0260
ULS: 2. D + L	-0.0000	1.6330	-0.0368	-0.1216	0.0202	0.0260
ULS: 3. D + (S or Lr or R)	-0.0000	4.8923	-0.1356	-0.4484	0.0748	0.0384
ULS: 3. D + (S or Lr or R)	-0.0000	1.6330	-0.0368	-0.1216	0.0202	0.0260
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	4.0775	-0.1109	-0.3667	0.0611	0.0353
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.6330	-0.0368	-0.1216	0.0202	0.0260
ULS: 5b. D + 0.7E	-0.0000	1.6330	-0.0368	-0.1216	0.0202	0.0260
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	4.0775	-0.1109	-0.3667	0.0611	0.0353

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 8. 0.6D + 0.7E	-0.0000	0.9798	-0.0221	-0.0730	0.0121	0.0156
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3563	3.9822	-0.1188	-0.3872	0.1420	14.5917
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3563	3.9822	-0.1188	-0.3872	0.1420	14.5917
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1626	-0.3807	0.0333	0.1046	-0.0842	-11.8858
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9688	-0.0450	0.0221	0.0686	-0.0675	-14.6515
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0173	5.8394	-0.1724	-0.5659	0.1525	10.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0173	5.8394	-0.1724	-0.5659	0.1525	10.9595
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8719	2.5672	-0.0583	-0.1971	-0.0171	-8.8986
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7266	2.8190	-0.0667	-0.2241	-0.0046	-10.9728
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0173	3.3949	-0.0983	-0.3208	0.1116	10.9503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0173	3.3949	-0.0983	-0.3208	0.1116	10.9503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8719	0.1228	0.0158	0.0480	-0.0581	-8.9079
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7266	0.3745	0.0074	0.0211	-0.0456	-10.9821
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3563	3.3290	-0.1041	-0.3385	0.1339	14.5813
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3563	3.3290	-0.1041	-0.3385	0.1339	14.5813
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1626	-1.0338	0.0480	0.1532	-0.0922	-11.8962
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9688	-0.6982	0.0369	0.1173	-0.0756	-14.6618

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value
Axial	9.1322
Shear X	-2.2606
Shear Z	-0.2707
Moment X	-0.8940
Moment Z	25.0964

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
Axial	5.8394
Shear X	-1.3563
Shear Z	-0.1724
Moment X	-0.5659
Moment Z	14.6618

Project Details

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

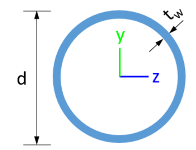
User Name: sales@mtsolar.us
Project Name: Dushane-RevA
Unit System: imperial

Design Input Information

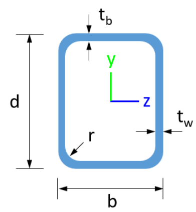
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

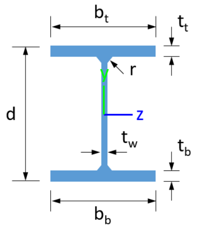
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
7	6in Pipe Sch 40	6.63	0.28					



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								
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

7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	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_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	7	21.98	21.98	10.47	-	300	200	1
2	5	2.00	2.00	2.00	-	300	200	1
3	16	1.42	1.42	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.16,1.18,1.18,1.17,1.17	300	200	1
4	16	3.75	3.75	3.75	1.69,1.68,1.69,1.67,1.69,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.65,1.79,1.67,1.67,1.68,1.67,1.68,1.68,1.61,1.71,1.67,1.67,1.66,1.14	300	200	1
5	16	2.33	2.33	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.65,1.67,1.67,1.66,1.66	300	200	1
6	16	1.42	1.42	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.15,1.17,1.18,1.18,1.18,1.18	300	200	1
7	16	2.33	2.33	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
8	19	2.05	2.05	2.05	1.27,1.27,1.27,1.27,1.27,1.27,1.26,1.26,1.25,1.32,1.26,1.26,1.26,1.73,1.27,1.27,1.27,1.28,1.26,1.26,1.25,1.32,1.26,1.26,1.26,1.09	300	200	1
9	2	4.00	4.00	4.00	-	300	200	1
10	16	3.75	3.75	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.86,1.67,1.67,1.68,1.67,1.68,1.68,1.62,1.71,1.67,1.67,1.66,1.53	300	200	1
11	19	2.05	2.05	2.05	1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.25,1.28,1.27,1.27,1.26,1.28,1.27,1.27,1.27,1.27,1.27,1.27,1.25,1.28,1.27,1.27,1.26,1.28	300	200	1
12	5	1.23	1.23	1.23	-	300	200	1
13	19	1.75	1.75	1.75	2.14,2.14,2.14,2.14,2.14,2.14,2.14,2.14,2.11,2.32,2.14,2.14,2.13,2.24,2.14,2.14,2.15,2.14,2.14,2.14,2.12,2.32,2.14,2.14,2.13,2.23	300	200	1
14	19	1.75	1.75	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
15	19	2.00	2.00	2.00	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	1
16	19	2.00	2.00	2.00	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	1
17	19	4.00	4.00	4.00	1.41,1.41,1.41,1.41,1.41,1.41,1.42,1.42,1.49,1.58,1.43,1.43,1.45,1.52,1.42,1.42,1.40,1.36,1.42,1.42,1.49,1.58,1.43,1.43,1.44,1.51	300	200	1
18	19	1.75	1.75	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
19	19	4.00	4.00	4.00	1.39,1.40,1.39,1.40,1.39,1.39,1.41,1.41,1.47,1.60,1.41,1.41,1.43,1.43,1.41,1.41,1.39,1.42,1.41,1.41,1.48,1.61,1.41,1.41,1.43,1.21	300	200	1
20	19	1.75	1.75	1.75	2.14,2.14,2.14,2.14,2.14,2.14,2.13,2.13,2.11,2.28,2.13,2.13,2.13,1.31,2.14,2.14,2.15,2.18,2.13,2.13,2.12,2.27,2.13,2.13,2.13,1.17	300	200	1
21	19	4.00	4.00	4.00	1.41,1.41,1.41,1.41,1.41,1.41,1.42,1.42,1.48,1.58,1.42,1.42,1.44,1.51,1.42,1.42,1.40,1.36,1.42,1.42,1.49,1.58,1.43,1.43,1.44,1.51	300	200	1
22	19	1.75	1.75	1.75	2.14,2.14,2.14,2.14,2.14,2.14,2.14,2.14,2.12,2.32,2.14,2.14,2.13,2.24,2.14,2.14,2.15,2.14,2.14,2.14,2.12,2.32,2.14,2.14,2.13,2.23	300	200	1
23	19	4.00	4.00	4.00	1.39,1.40,1.39,1.40,1.39,1.39,1.41,1.41,1.48,1.60,1.41,1.41,1.43,1.42,1.41,1.41,1.39,1.42,1.41,1.41,1.48,1.60,1.41,1.41,1.43,1.20	30	20	1

24	19	1.75	1.75	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
25	5	0.77	0.77	0.77	-	300	200	1
26	5	1.23	1.23	1.23	-	300	200	1
101	7	21.98	21.98	10.47	-	300	200	1
102	5	0.77	0.77	0.77	-	300	200	1
103	16	1.42	1.42	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.17,1.18,1.18,1.18,1.18	300	200	1
104	16	3.75	3.75	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.66,1.86,1.67,1.67,1.68,1.67,1.68,1.68,1.62,1.71,1.67,1.67,1.66,1.53	300	200	1
105	16	2.33	2.33	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
106	16	1.42	1.42	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.15,1.16,1.18,1.18,1.17,1.17	300	200	1
107	16	2.33	2.33	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.63,1.65,1.67,1.67,1.66,1.66	300	200	1
108	19	2.00	2.00	2.00	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	1
109	2	4.00	4.00	4.00	-	300	200	1
110	16	3.75	3.75	3.75	1.69,1.68,1.69,1.67,1.69,1.69,1.67,1.67,1.64,1.69,1.67,1.67,1.65,1.79,1.67,1.67,1.68,1.67,1.68,1.68,1.61,1.71,1.67,1.67,1.66,1.13	300	200	1
111	19	2.00	2.00	2.00	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	1
112	5	2.00	2.00	2.00	-	300	200	1
113	19	1.75	1.75	1.75	1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48,1.48	300	200	1
114	19	1.75	1.75	1.75	2.14,2.14,2.14,2.14,2.14,2.14,2.13,2.13,2.11,2.28,2.13,2.13,2.13,1.30,2.14,2.14,2.15,2.18,2.13,2.13,2.12,2.27,2.13,2.13,2.13,1.16	300	200	1
115	19	7.45	7.45	7.45	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.08,1.08,1.08	300	200	1
116	19	7.45	7.45	7.45	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.08,1.08,2.41,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.09,1.08,1.08,1.08,1.03	300	200	1

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	251.16	91.39	42.30	42.30	75.35	75.35
2	198.33	194.54	21.95	21.95	59.50	59.50
3	116.10	114.47	15.79	11.10	42.08	23.28
4	116.10	105.13	15.79	11.10	42.08	23.28
5	116.10	111.72	15.79	11.10	42.08	23.28
6	116.10	114.47	15.79	11.10	42.08	23.28
7	116.10	111.72	15.79	11.10	42.08	23.28
8	133.20	122.14	32.87	6.12	40.24	43.62
9	66.48	49.90	3.82	3.82	19.94	19.94

10	116.10	105.13	15.79	11.10	42.08	23.28
11	133.20	122.14	32.87	6.12	40.24	43.62
12	198.33	196.88	21.95	21.95	59.50	59.50
13	133.20	123.94	32.87	6.12	40.24	43.62
14	133.20	123.94	32.87	6.12	40.24	43.62
15	133.20	122.46	32.87	6.12	40.24	43.62
16	133.20	122.46	32.87	6.12	40.24	43.62
17	133.20	104.94	32.87	6.12	40.24	43.62
18	133.20	123.94	32.87	6.12	40.24	43.62
19	133.20	104.94	32.87	6.12	40.24	43.62
20	133.20	123.94	32.87	6.12	40.24	43.62
21	133.20	104.94	32.87	6.12	40.24	43.62
22	133.20	123.94	32.87	6.12	40.24	43.62
23	133.20	104.94	32.87	6.12	40.24	43.62
24	133.20	123.94	32.87	6.12	40.24	43.62
25	198.33	197.77	21.95	21.95	59.50	59.50
26	198.33	196.88	21.95	21.95	59.50	59.50
101	251.16	91.39	42.30	42.30	75.35	75.35
102	198.33	197.77	21.95	21.95	59.50	59.50
103	116.10	114.47	15.79	11.10	42.08	23.28
104	116.10	105.13	15.79	11.10	42.08	23.28
105	116.10	111.72	15.79	11.10	42.08	23.28
106	116.10	114.47	15.79	11.10	42.08	23.28
107	116.10	111.72	15.79	11.10	42.08	23.28
108	133.20	122.46	32.87	6.12	40.24	43.62
109	66.48	49.90	3.82	3.82	19.94	19.94
110	116.10	105.13	15.79	11.10	42.08	23.28
111	133.20	122.46	32.87	6.12	40.24	43.62
112	198.33	194.54	21.95	21.95	59.50	59.50
113	133.20	123.94	32.87	6.12	40.24	43.62
114	133.20	123.94	32.87	6.12	40.24	43.62
115	133.20	58.22	24.87	6.12	40.24	43.62
116	133.20	58.22	23.72	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	0.100	0.593	0.046	0.030	0.004	0.648	#13	0.587	Not Required	Pass
2	0.001	0.288	0.100	0.066	0.019	0.363	#21	0.081	Not Required	Pass
3	0.006	0.447	0.032	0.044	0.007	0.479	#21	0.070	Not Required	Pass
4	0.006	0.443	0.060	0.045	0.015	0.503	#21	0.123	Not Required	Pass
5	0.006	0.277	0.047	0.044	0.011	0.284	#21	0.115	Not Required	Pass
6	0.009	0.526	0.084	0.053	0.023	0.614	#21	0.070	Not Required	Pass
7	0.009	0.326	0.112	0.052	0.028	0.354	#21	0.115	Not Required	Pass
8	0.002	0.110	0.078	0.033	0.013	0.189	#21	0.146	Not Required	Pass
9	0.002	0.040	0.056	0.001	0.003	0.094	#21	0.313	Not Required	Pass
10	0.010	0.523	0.103	0.053	0.023	0.595	#21	0.123	Not Required	Pass
11	0.003	0.110	0.077	0.033	0.013	0.187	#21	0.146	Not Required	Pass
12	0.000	0.376	0.112	0.083	0.020	0.460	#21	0.050	Not Required	Pass
13	0.003	0.044	0.260	0.046	0.017	0.303	#21	0.125	Not Required	Pass
14	0.000	0.060	0.135	0.026	0.010	0.195	#21	Not Required	Not Required	Pass
15	0.000	0.017	0.038	0.014	0.005	0.055	#21	Not Required	Not Required	Pass
16	0.000	0.017	0.038	0.014	0.005	0.055	#21	Not Required	Not Required	Pass

17	0.002	0.077	0.058	0.020	0.006	0.128	#21	0.190	Not Required	Pass
18	0.000	0.060	0.135	0.026	0.010	0.195	#21	Not Required	Not Required	Pass
19	0.003	0.078	0.083	0.020	0.007	0.136	#21	0.286	Not Required	Pass
20	0.002	0.045	0.256	0.046	0.017	0.297	#21	0.125	Not Required	Pass
21	0.002	0.077	0.058	0.020	0.006	0.128	#21	0.190	Not Required	Pass
22	0.003	0.044	0.260	0.046	0.017	0.303	#21	0.125	Not Required	Pass
23	0.003	0.078	0.083	0.020	0.007	0.136	#21	0.286	Not Required	Pass
24	0.000	0.060	0.135	0.026	0.010	0.195	#21	Not Required	Not Required	Pass
25	0.000	0.099	0.061	0.083	0.020	0.160	#21	0.031	Not Required	Pass
26	0.000	0.376	0.112	0.083	0.020	0.460	#21	0.050	Not Required	Pass
101	0.100	0.593	0.046	0.030	0.004	0.648	#13	0.587	Not Required	Pass
102	0.000	0.099	0.061	0.083	0.020	0.160	#21	0.031	Not Required	Pass
103	0.009	0.526	0.084	0.053	0.023	0.614	#21	0.070	Not Required	Pass
104	0.010	0.523	0.103	0.053	0.023	0.595	#21	0.123	Not Required	Pass
105	0.009	0.326	0.112	0.052	0.028	0.354	#21	0.115	Not Required	Pass
106	0.006	0.447	0.032	0.044	0.007	0.479	#21	0.070	Not Required	Pass
107	0.006	0.277	0.047	0.044	0.011	0.284	#21	0.115	Not Required	Pass
108	0.000	0.017	0.038	0.014	0.005	0.055	#21	Not Required	Not Required	Pass
109	0.002	0.040	0.056	0.001	0.003	0.094	#21	0.313	Not Required	Pass
110	0.006	0.443	0.060	0.045	0.015	0.503	#21	0.123	Not Required	Pass
111	0.000	0.017	0.038	0.014	0.005	0.055	#21	Not Required	Not Required	Pass
112	0.001	0.288	0.100	0.066	0.019	0.363	#21	0.081	Not Required	Pass
113	0.000	0.060	0.135	0.026	0.010	0.195	#21	Not Required	Not Required	Pass
114	0.002	0.045	0.256	0.046	0.017	0.297	#21	0.125	Not Required	Pass
115	0.007	0.186	0.147	0.033	0.013	0.336	#21	0.532	Not Required	Pass
116	0.002	0.187	0.148	0.033	0.013	0.335	#21	0.532	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/comresssion)
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

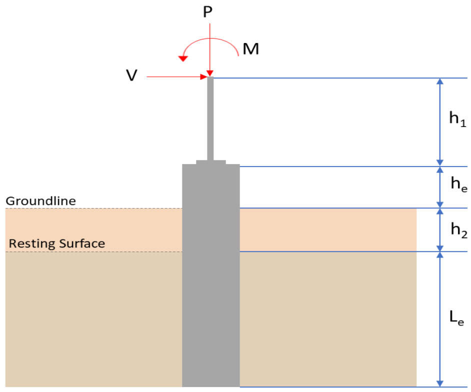
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 5.25\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.839	9.132
V_x (kip)	-1.356	-2.261
V_z (kip)	0.172	0.271
M_x (kipft)	0.566	0.894
M_z (kipft)	14.662	25.096

Material Properties

$f'_{ck} = 3\ \mathrm{ksi}$ - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \mathrm{ft}) + (0\ \mathrm{ft}) + (0\ \mathrm{ft})$$

$$H = 0\ \mathrm{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.356\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.21592\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.662\ \mathrm{kipft}) + ((-1.356\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 2.3347\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 4.9657\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\, b}$$

$$H_o = \frac{(0.172\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.027389\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$

$$M_o = \frac{(0.566\ \mathrm{kipft}) + ((0.172\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.090127\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

$$L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,z} = 2.2137\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = MAX[(4.9657\ \mathrm{ft}), (2.2137\ \mathrm{ft})]$$

$$L_{e,req} = 4.966\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 5.25\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.966\ \mathrm{ft})}{(5.25\ \mathrm{ft})}$$

$$Ratio = 0.9459$$

Status: **PASS**
Ratio: **0.950**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48\ \mathrm{in}) \times (48\ \mathrm{in})$$

$$A = 16\ \mathrm{ft}^2$$

q - End-bearing pressure

$$q = \frac{P_v}{A}$$

$$q = \frac{(5.839\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.36494\ \mathrm{kip/ft}^2$$

$q = 0.36494\ \mathrm{kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

$$Ratio = \frac{q}{q_a}$$

$$Ratio = \frac{\left(0.36494\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.18247$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.3125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.21592\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.3347\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3347\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (2.3347\ \mathrm{kipft/ft})) + (4 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.607\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3347\ \mathrm{kipft/ft})) + (3 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{\ 2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (2.3347\ \mathrm{kipft/ft})) + (2 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = 0.20255\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.3347\ \mathrm{kipft/ft})) + ((-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.7697\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.607\ \mathrm{ft})}{2}$$

$$p_a = 0.27052\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.20255\ \mathrm{kip/ft}^2\right)}{\left(0.27052\ \mathrm{kip/ft}^2\right)}$$

$$Ratio = 0.74873$$

Status: **PASS**
Ratio: **0.750**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.25\ \mathrm{ft})$$

$$p_s = 0.7875\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.7697\ \mathrm{kip/ft^2})}{(0.7875\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.9774$$

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

Considering z-direction:

$H_o = 0.027389\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.090127\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.090127\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (0.090127\ \mathrm{kipft/ft})) + (4 \times (0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.7255\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.090127\ \mathrm{kipft/ft})) + (3 \times (0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{\ 2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (0.090127\ \mathrm{kipft/ft})) + (2 \times (0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = 0.030581\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.090127\ \mathrm{kipft/ft})) + ((0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.07054\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.7255\ \mathrm{ft})}{2}$$

$$p_a = 0.27941\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.030581\ \mathrm{kip/ft^2})}{(0.27941\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.10945$$

Status: **PASS**
Ratio: **0.110**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.25\ \mathrm{ft})$$

$$p_s = 0.7875\ \mathrm{kip/ft^2}$$

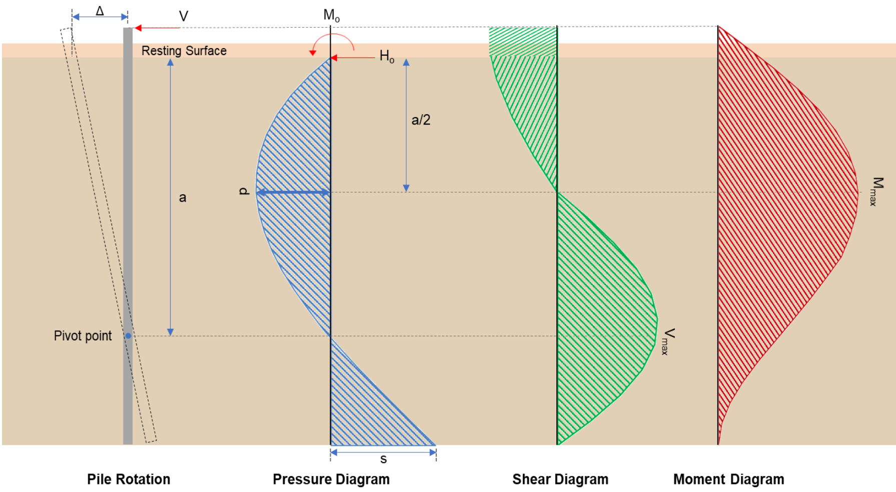
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.07054\ \mathrm{kip/ft^2})}{(0.7875\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.089575$$

Status: **PASS**
Ratio: **0.090**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.261\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.36003\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.096\ \mathrm{kipft}) + ((-2.261\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 3.9962\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.9962\ \mathrm{kipft/ft})}{(-0.36003\ \mathrm{kip/ft})}$$

$$E = 11.1\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9962\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.36003\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (3.9962\ \mathrm{kipft/ft})) + (4 \times (-0.36003\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.6049\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.36003\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.1\ \mathrm{ft})}{(5.25\ \mathrm{ft})} + 3\right) \times \left(\frac{(3.6049\ \mathrm{ft})}{(5.25\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.1\ \mathrm{ft})}{(5.25\ \mathrm{ft})} + 2\right) \times \left(\frac{(3.6049\ \mathrm{ft})}{(5.25\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.3389\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.36003\ \text{kip/ft}) \times (48\ \text{in}) \times (5.25\ \text{ft})) \times \left[\left(\frac{(11.1\ \text{ft})}{(5.25\ \text{ft})} + \frac{(3.6049\ \text{ft})}{2 \times (5.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (11.1\ \text{ft})}{(5.25\ \text{ft})} + 3\right) \times \left(\frac{(3.6049\ \text{ft})}{2 \times (5.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.1\ \text{ft})}{(5.25\ \text{ft})} + 2\right) \times \left(\frac{(3.6049\ \text{ft})}{2 \times (5.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 15.951\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\,b}$$

$$H_o = \frac{(0.271\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.043153\ \text{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\,b}$$

$$M_o = \frac{(0.894\ \text{kipft}) + ((0.271\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.14236\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14236\ \text{kipft/ft})}{(0.043153\ \text{kip/ft})}$$

$$E = 3.2989\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14236\ \text{kipft/ft}) \times (5.25\ \text{ft})) + \left(3 \times (0.043153\ \text{kip/ft}) \times (5.25\ \text{ft})^2\right)}{(6 \times (0.14236\ \text{kipft/ft})) + (4 \times (0.043153\ \text{kip/ft}) \times (5.25\ \text{ft}))}$$

$$a = 3.7252\ \text{ft}$$

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

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

$$V_{max} = ((0.043153\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2989\ \text{ft})}{(5.25\ \text{ft})} + 3\right) \times \left(\frac{(3.7252\ \text{ft})}{(5.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.2989\ \text{ft})}{(5.25\ \text{ft})} + 2\right) \times \left(\frac{(3.7252\ \text{ft})}{(5.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.30655\ \text{kip}$$

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

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

$$M_{max} = ((0.043153\ \text{kip/ft}) \times (48\ \text{in}) \times (5.25\ \text{ft})) \times \left[\left(\frac{(3.2989\ \text{ft})}{(5.25\ \text{ft})} + \frac{(3.7252\ \text{ft})}{2 \times (5.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.2989\ \text{ft})}{(5.25\ \text{ft})} + 3\right) \times \left(\frac{(3.7252\ \text{ft})}{2 \times (5.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.2989\ \text{ft})}{(5.25\ \text{ft})} + 2\right) \times \left(\frac{(3.7252\ \text{ft})}{2 \times (5.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.72359 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.132 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -101.96 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-101.96 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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 $$\phi P_N = \phi\, 0.80\ \left[\left(0.85\, f'_{ck}\ \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (3\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 3183.4\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.132\text{ kip})}{(3183.4\text{ kip})}$$ $$Ratio = 0.0028686$$	Status: **PASS** Ratio: **0.000**

	Shear Strength (ACI 318-19, LRFD) **Parameters:** $b_w = 48\text{ in}$ - Effective width,	
22.5.2.2	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 $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$,	
22.5.5.1.1	$V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\,\lambda_s\,\sqrt{f'_{ck}}\,b_w\,d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,max} = 324.49\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$, $P = 9.132\text{ kip} \rightarrow 9132\text{ lbf}$,	
22.5.5.1.1(a)	$V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \frac{P}{6\,A_g}\right]\,b_w\,d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\text{ psi})} + \frac{(9132\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 131.01\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$,	
22.5.5.1.2	$V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \left(0.05\,f'_{ck}\right)\right]\,b_w\,d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000\text{ psi})} + (0.05 \times (3000\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 406.27\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(324.49\text{ kip}), (131.01\text{ kip}), (406.27\text{ kip})\right]$$ $$V_c = 131.01\text{ kip}$$	

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\text{ ksi} \rightarrow 3000\text{ psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$

$$V_{s,a} = 8 \times \sqrt{(3000\text{ psi})} \times (48\text{ in}) \times (38.4\text{ in})$$

$$V_{s,a} = 807.65\text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\text{ in})^2}{4}$$

$$A_v = 0.11045\text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(807.65\text{ kip}), (50.894\text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((131.01\text{ kip}) + (50.894\text{ kip}))$$

$$\phi V_n = 118.24\text{ kip}$$

Considering x-direction:

$V_{max} = 6.3389\text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.3389\text{ kip})}{(118.24\text{ kip})}$$

$$Ratio = 0.053611$$

Status: **PASS**
Ratio: **0.050**

Considering z-direction:

$V_{max} = 0.30655\text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.30655\text{ kip})}{(118.24\text{ kip})}$$

$$Ratio = 0.0025926$$

Status: **PASS**
Ratio: **0.000**

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: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2545.9\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(273.42\ \mathrm{kipft}), (2545.9\ \mathrm{kipft})]$$ $$\phi M_n = 273.42\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 15.951\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(15.951\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$ $$Ratio = 0.05834$$	Status: **PASS** Ratio: **0.060**
	Considering z-direction: $M_{max} = 0.72359\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.72359\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$ $$Ratio = 0.0026464$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

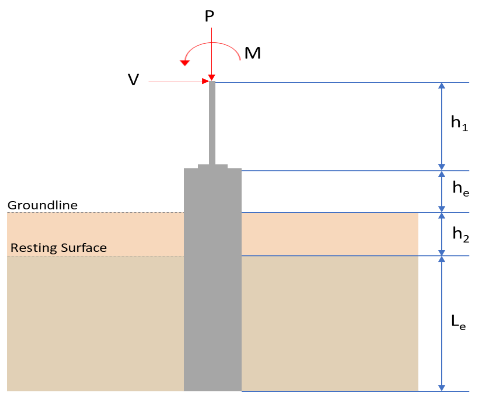
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 5.25$ ft - Total pile length
$h_1 = 0$ ft - Lateral load height from the top of the pile,
$h_2 = 0$ ft - Depth to resisting surface
$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.839	9.132
V_x (kip)	-1.356	-2.261
V_z (kip)	-0.172	-0.271
M_x (kipft)	-0.566	-0.894
M_z (kipft)	14.662	25.096

Material Properties

$f'_{ck} = 3$ ksi - Concrete strength,

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0\ \text{ft}) + (0\ \text{ft}) + (0\ \text{ft})$$

$$H = 0\ \text{ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.356\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.21592\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.662\ \mathrm{kipft}) + ((-1.356\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 2.3347\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 4.9657\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\, b}$$

$$H_o = \frac{(-0.172\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.027389\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$

$$M_o = \frac{(0.566\ \mathrm{kipft}) + ((-0.172\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.090127\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

$$L_z^3 - \left(14.14 \times \frac{H_o \times L_z}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$$

Solving the cubic equation:
$L_{e,z} = 1.6507\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

$L_{e,req}$ - Depth of pile required,

$$L_{e,req} = MAX[L_{e,x}, L_{e,z}]$$

$$L_{e,req} = MAX[(4.9657\ \mathrm{ft}), (1.6507\ \mathrm{ft})]$$

$$L_{e,req} = 4.966\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 5.25\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(4.966\ \mathrm{ft})}{(5.25\ \mathrm{ft})}$$

$$Ratio = 0.9459$$

Status: **PASS**
Ratio: **0.950**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

$$A = (48\ \mathrm{in}) \times (48\ \mathrm{in})$$

$$A = 16\ \mathrm{ft}^2$$

q - End-bearing pressure

$$q = \frac{P_v}{A}$$

$$q = \frac{(5.839\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.36494\ \mathrm{kip/ft}^2$$

$$q = 0.36494\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

$$Ratio = \frac{q}{q_a}$$

$$Ratio = \frac{(0.36494\ \mathrm{kip/ft^2})}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.18247$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.3125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.21592\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.3347\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.3347\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (2.3347\ \mathrm{kipft/ft})) + (4 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.607\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.3347\ \mathrm{kipft/ft})) + (3 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (2.3347\ \mathrm{kipft/ft})) + (2 \times (-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = 0.20255\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.3347\ \mathrm{kipft/ft})) + ((-0.21592\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.7697\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.607\ \mathrm{ft})}{2}$$

$$p_a = 0.27052\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.20255\ \mathrm{kip/ft^2})}{(0.27052\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.74873$$

Status: **PASS**
Ratio: **0.750**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.25\ \mathrm{ft})$$

$$p_s = 0.7875\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.7697\ \mathrm{kip/ft^2}\right)}{\left(0.7875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9774$$

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

Considering z-direction:

$H_o = -0.027389\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.090127\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.090127\ \mathrm{kipft/ft}) \times (5.25\ \mathrm{ft})) + \left(3 \times (-0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft})^2\right)}{(6 \times (0.090127\ \mathrm{kipft/ft})) + (4 \times (-0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))}$$

$$a = 3.7255\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.090127\ \mathrm{kipft/ft})) + (3 \times (-0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]^{\,2}}{(5.25\ \mathrm{ft})^2 \times [(3 \times (0.090127\ \mathrm{kipft/ft})) + (2 \times (-0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}$$

$$p = -0.0079447\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.090127\ \mathrm{kipft/ft})) + ((-0.027389\ \mathrm{kip/ft}) \times (5.25\ \mathrm{ft}))]}{(5.25\ \mathrm{ft})^2}$$

$$s = 0.007938\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.7255\ \mathrm{ft})}{2}$$

$$p_a = 0.27941\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0079447\ \mathrm{kip/ft^2}\right)}{\left(0.27941\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.028434$$

Status: **PASS**
Ratio: **-0.030**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.25\ \mathrm{ft})$$

$$p_s = 0.7875\ \mathrm{kip/ft^2}$$

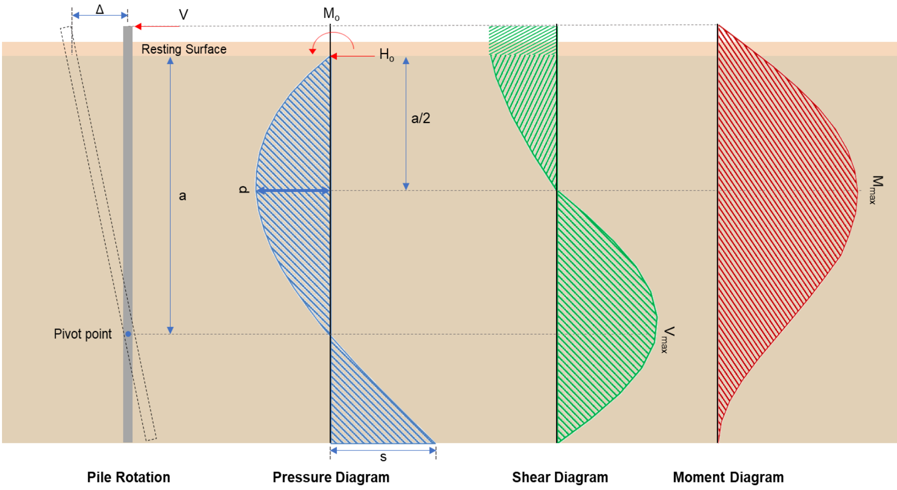
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.007938\ \mathrm{kip/ft^2}\right)}{\left(0.7875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.01008$$

Status: **PASS**
Ratio: **0.010**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{\left(-2.261\ \mathrm{kip}\right)}{1.57 \times \left(48\ \mathrm{in}\right)}$$

$$H_o = -0.36003\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{\left(25.096\ \mathrm{kipft}\right) + \left(\left(-2.261\ \mathrm{kip}\right) \times \left(0\ \mathrm{ft}\right)\right)}{1.57 \times \left(48\ \mathrm{in}\right)}$$

$$M_o = 3.9962\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{\left(3.9962\ \mathrm{kipft/ft}\right)}{\left(-0.36003\ \mathrm{kip/ft}\right)}$$

$$E = 11.1\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{\left(4 \times \left(3.9962\ \mathrm{kipft/ft}\right) \times \left(5.25\ \mathrm{ft}\right)\right) + \left(3 \times \left(-0.36003\ \mathrm{kip/ft}\right) \times \left(5.25\ \mathrm{ft}\right)^2\right)}{\left(6 \times \left(3.9962\ \mathrm{kipft/ft}\right)\right) + \left(4 \times \left(-0.36003\ \mathrm{kip/ft}\right) \times \left(5.25\ \mathrm{ft}\right)\right)}$$

$$a = 3.6049\ \mathrm{ft}$$

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

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

$$V_{max} = \left(\left(-0.36003\ \mathrm{kip/ft}\right) \times \left(48\ \mathrm{in}\right)\right) \times \left[1 - \left[3 \times \left(\frac{4 \times \left(11.1\ \mathrm{ft}\right)}{\left(5.25\ \mathrm{ft}\right)} + 3\right) \times \left(\frac{\left(3.6049\ \mathrm{ft}\right)}{\left(5.25\ \mathrm{ft}\right)}\right)^2\right] + \left[4 \times \left(\frac{3 \times \left(11.1\ \mathrm{ft}\right)}{\left(5.25\ \mathrm{ft}\right)} + 2\right) \times \left(\frac{\left(3.6049\ \mathrm{ft}\right)}{\left(5.25\ \mathrm{ft}\right)}\right)^3\right]\right]$$

$$V_{max} = 6.3389\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.36003 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(11.1 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6049 \text{ ft})}{2 \times (5.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (11.1 \text{ ft})}{(5.25 \text{ ft})} + 3\right) \times \left(\frac{(3.6049 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.1 \text{ ft})}{(5.25 \text{ ft})} + 2\right) \times \left(\frac{(3.6049 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 15.951 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{1.57\, b}$$

$$H_o = \frac{(-0.271 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.043153 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_x + (V_z\, H)}{1.57\, b}$$

$$M_o = \frac{(0.894 \text{ kipft}) + ((-0.271 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14236 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14236 \text{ kipft/ft})}{(-0.043153 \text{ kip/ft})}$$

$$E = 3.2989 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14236 \text{ kipft/ft}) \times (5.25 \text{ ft})) + \left(3 \times (-0.043153 \text{ kip/ft}) \times (5.25 \text{ ft})^2\right)}{(6 \times (0.14236 \text{ kipft/ft})) + (4 \times (-0.043153 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7252 \text{ ft}$$

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

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

$$V_{max} = ((-0.043153 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2989 \text{ ft})}{(5.25 \text{ ft})} + 3\right) \times \left(\frac{(3.7252 \text{ ft})}{(5.25 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.2989 \text{ ft})}{(5.25 \text{ ft})} + 2\right) \times \left(\frac{(3.7252 \text{ ft})}{(5.25 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.30655 \text{ kip}$$

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

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

$$M_{max} = ((-0.043153 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(3.2989 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7252 \text{ ft})}{2 \times (5.25 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.2989 \text{ ft})}{(5.25 \text{ ft})} + 3\right) \times \left(\frac{(3.7252 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.2989 \text{ ft})}{(5.25 \text{ ft})} + 2\right) \times \left(\frac{(3.7252 \text{ ft})}{2 \times (5.25 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 0.72359 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
Table 22.4.2.1 $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$
22.4.2.2, 10.6.1.1 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.132 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (3 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -101.96 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-101.96 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{\left(4.1472 \text{ in}^2\right)}{\left(0.3068 \text{ in}^2\right)}$$

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (3\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$ $$\phi P_N = 3183.4\ \mathrm{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.132\ \mathrm{kip})}{(3183.4\ \mathrm{kip})}$$ $$Ratio = 0.0028686$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\ \mathrm{in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\ \mathrm{in})$$ $$d = 38.4\ \mathrm{in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1+\frac{(38.4\ \mathrm{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,max} = 324.49\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $P = 9.132\ \mathrm{kip} \rightarrow 9132\ \mathrm{lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + \frac{(9132\ \mathrm{lbf})}{6 \times \left(2304\ \mathrm{in}^2\right)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,a} = 131.01\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \rightarrow 3000\ \mathrm{psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000\ \mathrm{psi})} + (0.05 \times (3000\ \mathrm{psi}))\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,b} = 406.27\ \mathrm{kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(324.49\ \mathrm{kip}), (131.01\ \mathrm{kip}), (406.27\ \mathrm{kip})\right]$$ $$V_c = 131.01\ \mathrm{kip}$$	

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3\ \mathrm{ksi} \to 3000\ \mathrm{psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

$$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$

$$V_{s,a} = 8 \times \sqrt{(3000\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{s,a} = 807.65\ \mathrm{kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi\, d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$

$$A_v = 0.11045\ \mathrm{in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(807.65\ \mathrm{kip}), (50.894\ \mathrm{kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((131.01\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 118.24\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 6.3389\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(6.3389\ \mathrm{kip})}{(118.24\ \mathrm{kip})}$$

$$Ratio = 0.053611$$

Status: **PASS**
Ratio: **0.050**

Considering z-direction:

$V_{max} = 0.30655\ \mathrm{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.30655\ \mathrm{kip})}{(118.24\ \mathrm{kip})}$$

$$Ratio = 0.0025926$$

Status: **PASS**
Ratio: **0.000**

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 273.423kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (3\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2545.9\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(273.42\ \mathrm{kipft}), (2545.9\ \mathrm{kipft})]$$ $$\phi M_n = 273.42\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 15.951\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(15.951\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$ $$Ratio = 0.05834$$	Status: **PASS** Ratio: **0.060**
	Considering z-direction: $M_{max} = 0.72359\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.72359\ \mathrm{kipft})}{(273.42\ \mathrm{kipft})}$$ $$Ratio = 0.0026464$$	Status: **PASS** Ratio: **0.000**