Project Details

Project Name: MTSOLAR_2GDH4I857I61-Clinton-Portrait

Location: 2141 Sunnyside Ave, El Cajon, CA 92019, USA

Unique ID: 2P-22.5-8TOP-XD-45-L-3Hx10W-3575

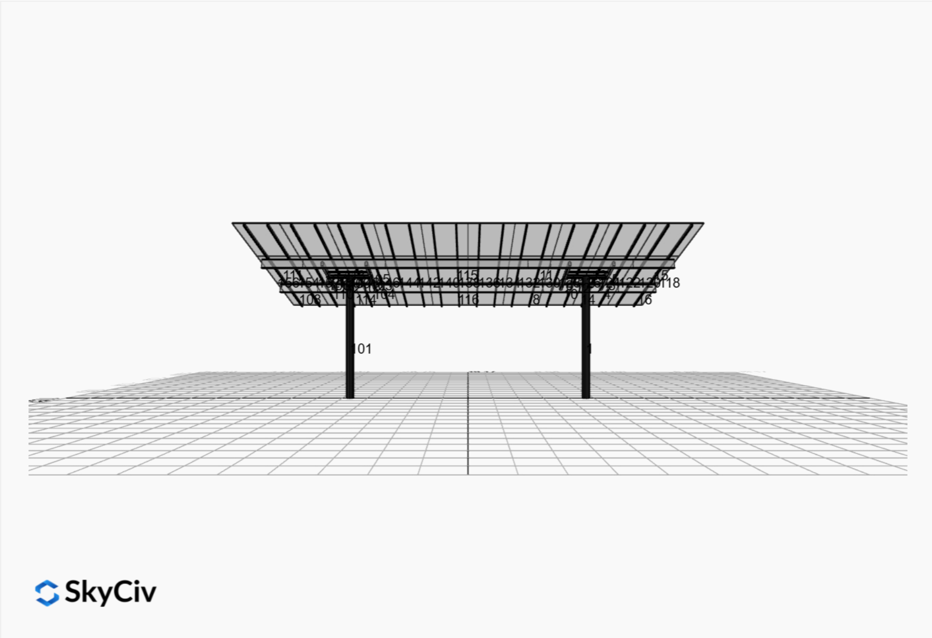
Dealer: ______________________________

Date: Mon Oct 07 2024

Number of Modules: 30

Number of Poles: 2

Date Sold: ______________________________



Array Dimensions N/S	23.88 ft
Array Dimensions E/W	38.33 ft
Winter Tilt Angle	15
Front Edge Clearance	8.5 ft

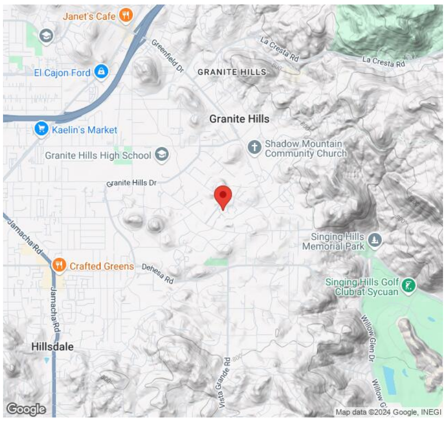
MT Solar Bill of Materials (2P-22.5-8TOP-XD-45-L-3Hx10W-3575)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	2
MTS-HF-XD	H-Frame Assembly-XD	2
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	10

Rail Bill of Materials

Part	Qty
Rails (285in)	20
Rail Attachment	40
Module Mid Clamp	40
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: 2141 Sunnyside Ave, El Cajon, CA 92019, USA

Array Specification

Duty Classification:	XD
Module Width:	95.00 in
Module Length:	45.00in
Number of Rows:	3
Number of Columns:	10
Total Number of Modules:	30
Winter Tilt Angle:	15
Front Edge Clearance:	8.5
Total Array Height at Tilt:	14.68 ft
Total Frame Length:	37.50 ft
Frame Weight:	3452 lbs
Array Dimensions N/S:	23.88 ft
Array Dimensions E/W:	38.33 ft
Rail Length:	286.50 in
Rail Spacing:	1.92 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.59 ft
Number of Poles:	2
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.50 ft Pile 2: 6.50 ft
Foundation Volume:	$7.704\ y^3$

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	2141 Sunnyside Ave, El Cajon, CA 92019, USA
Wind Speed:	96 mph
Snow Load:	0 psf

Design Disclaimer

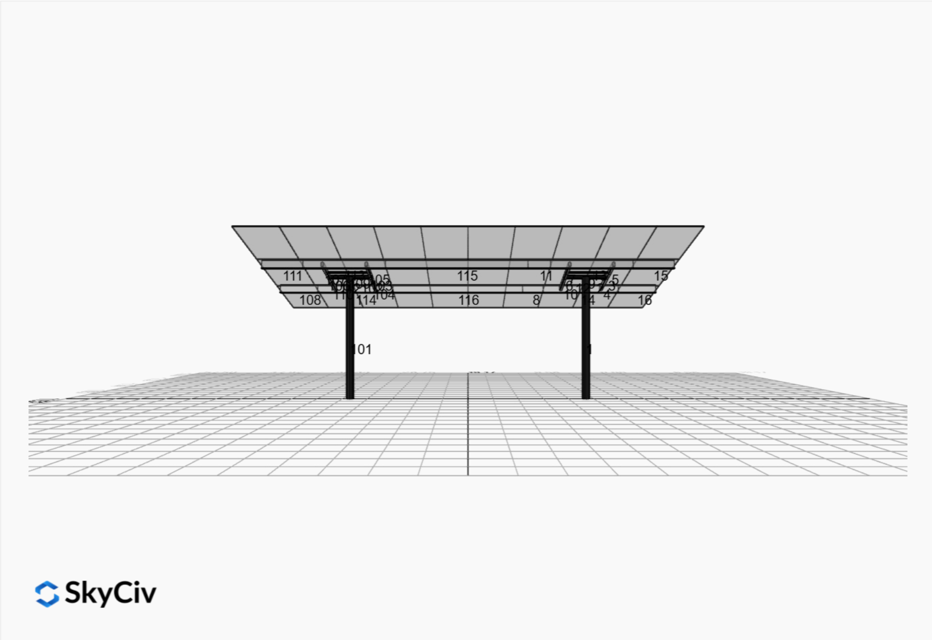
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

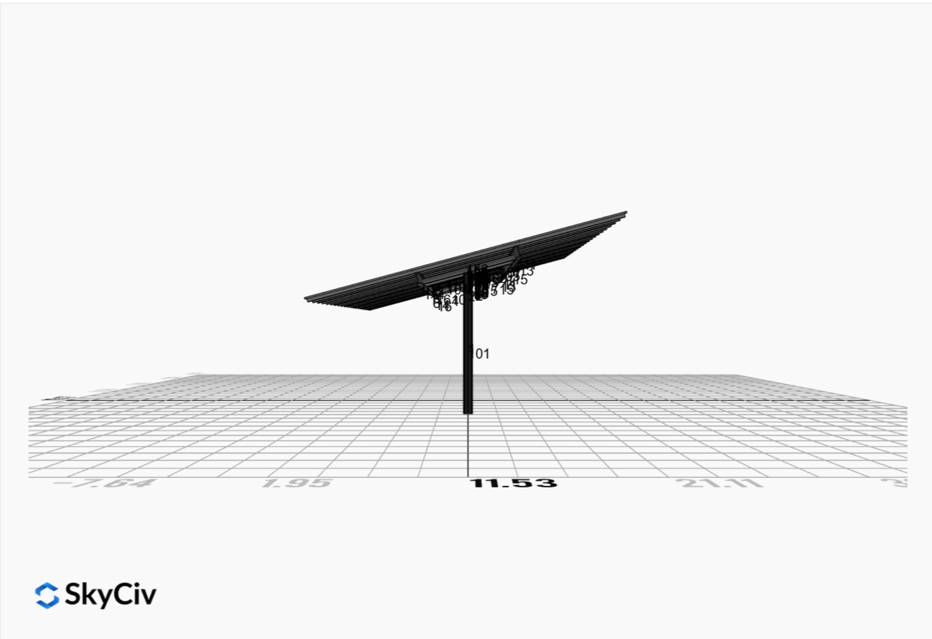
AutoDesigner Input

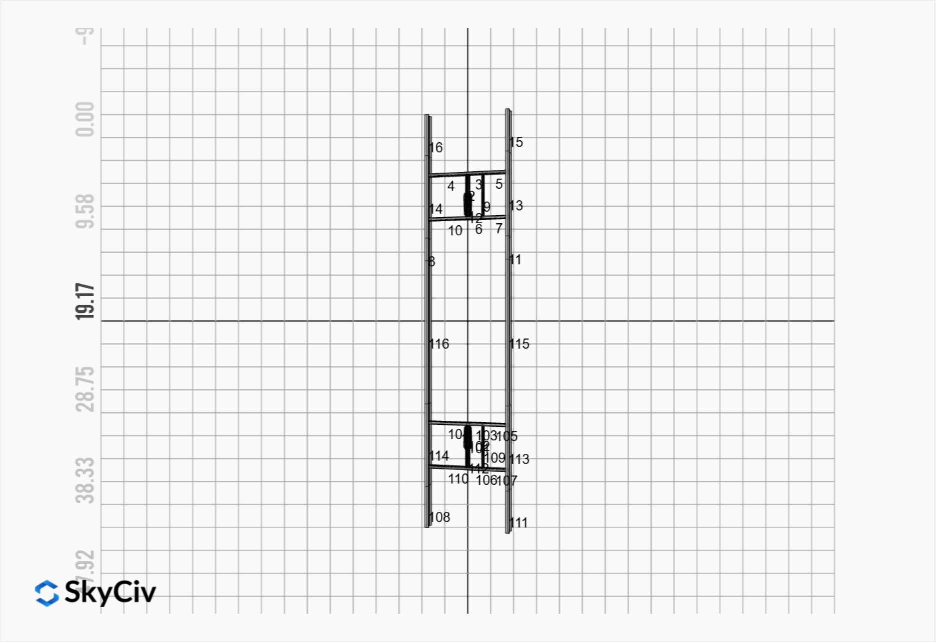
{"wind_speed_override":96,"snow_load_override":null,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"Clinton Johnson","designer_email":"clinton@johnsonsolar.com","designer_phone":"6199097337","project_id":"MTSOLAR_2GDH4I857I61-Clinton-Portrait","site_address":"2141 Sunnyside Ave, El Cajon, CA 92019, USA","module_width":95,"module_length":45,"number_rows":"3","number_columns":10,"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":15,"ground_clearance":8.5,"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":48,"check_rails":false}

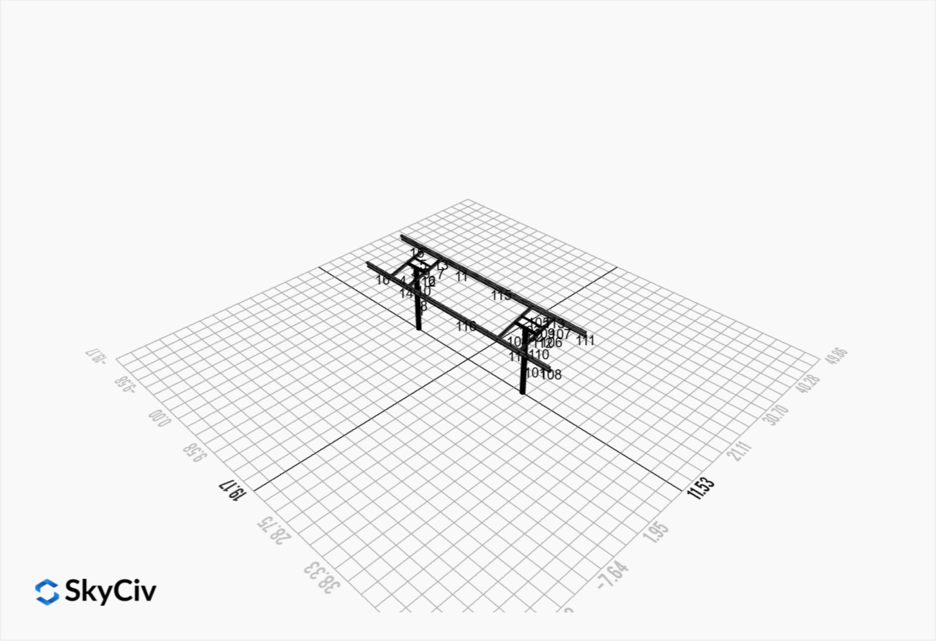
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only






SkyCiv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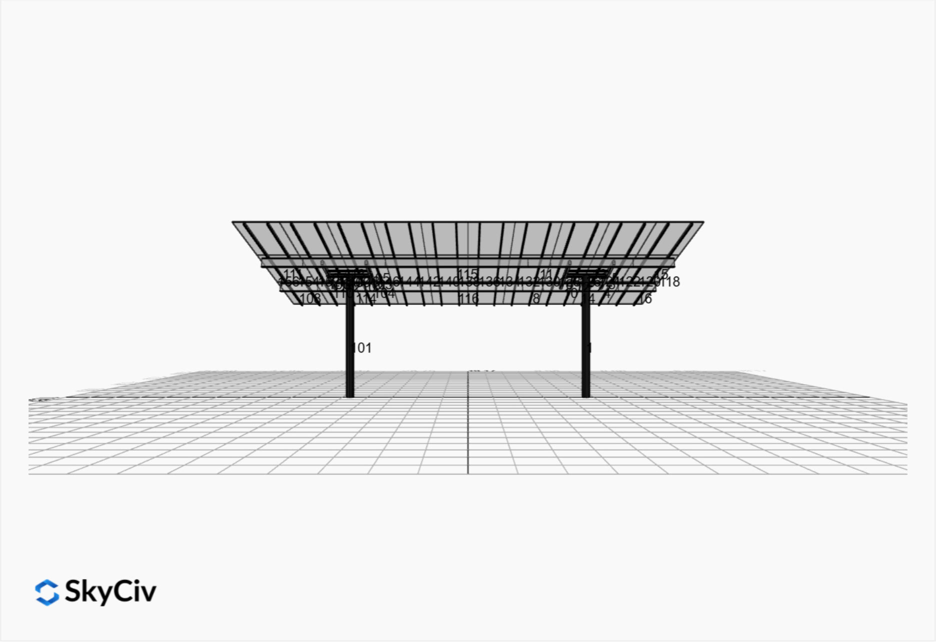
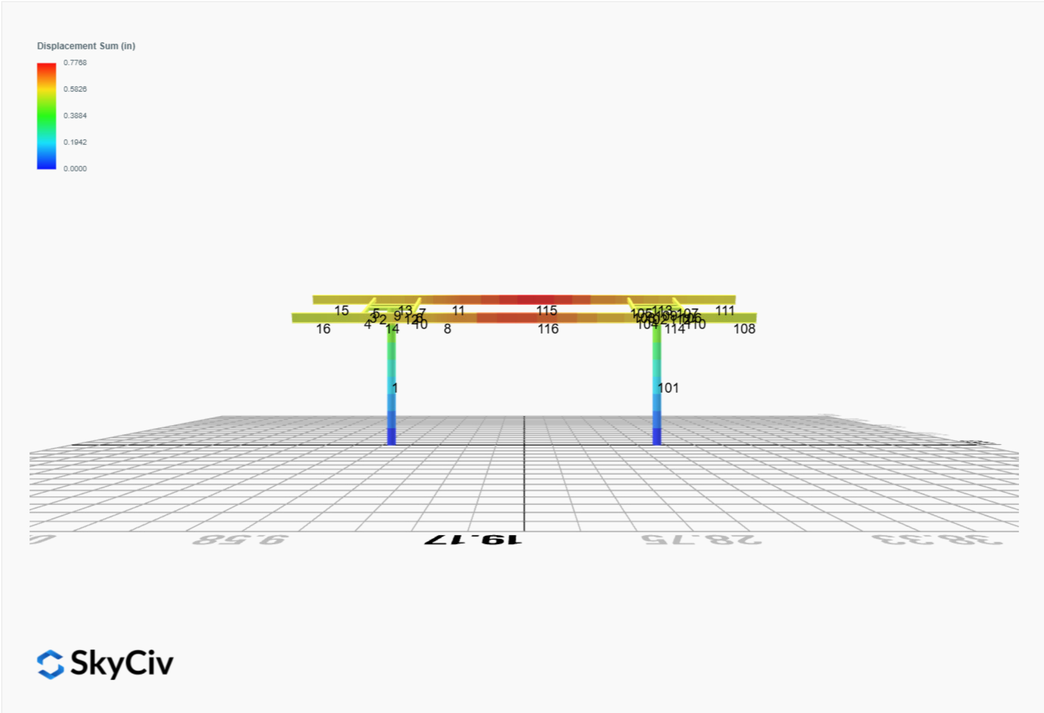
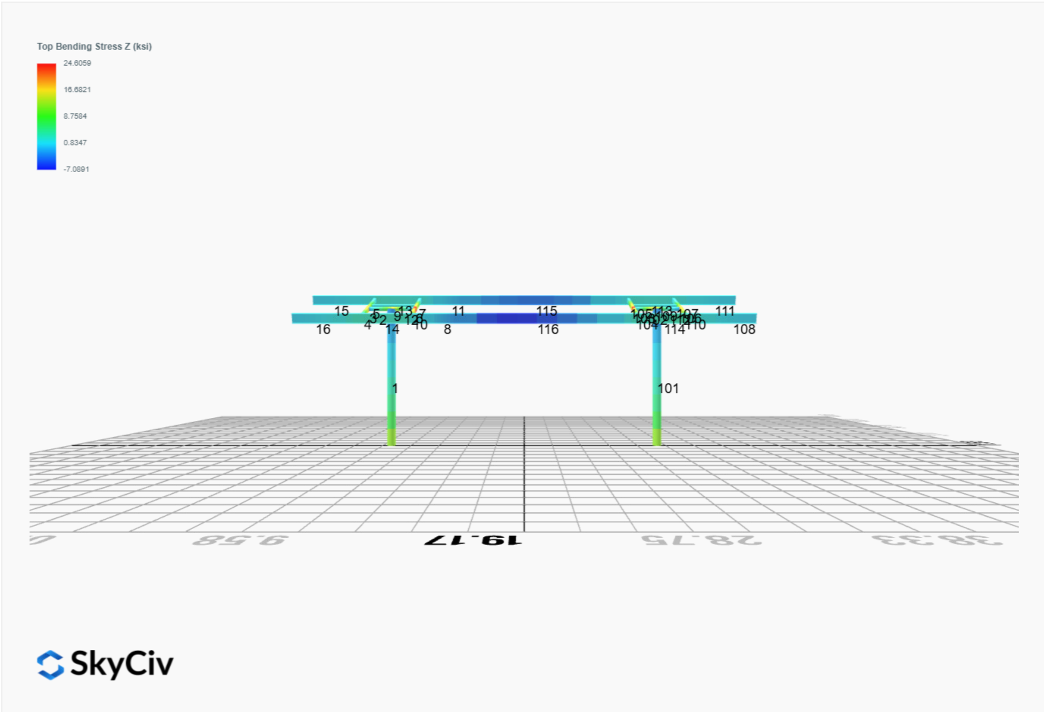
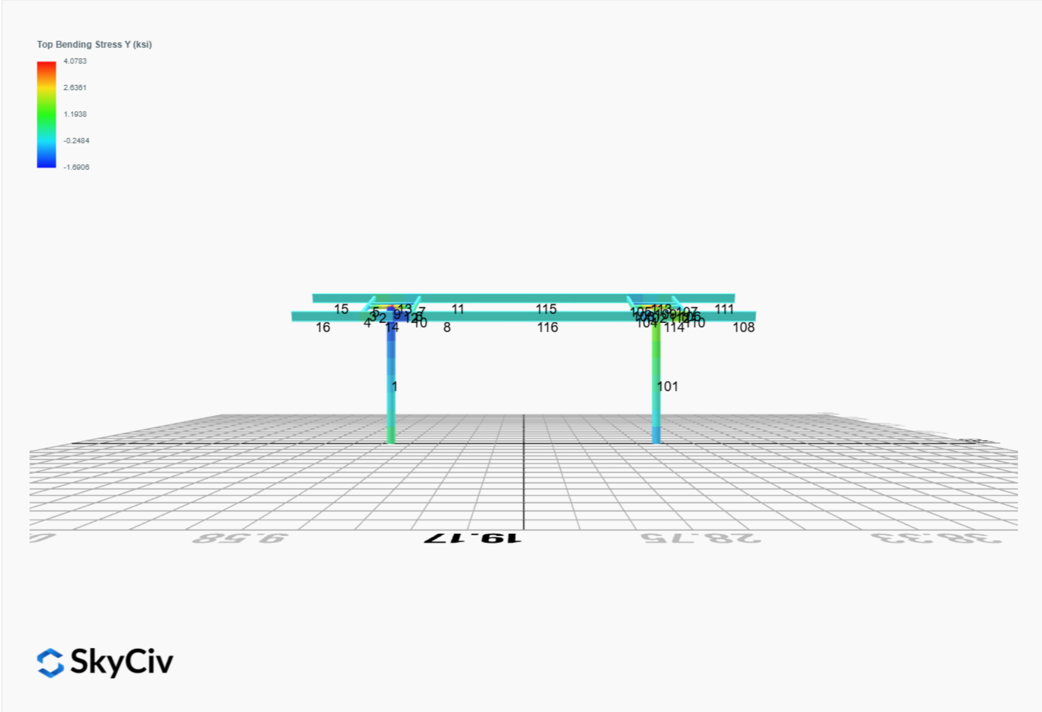
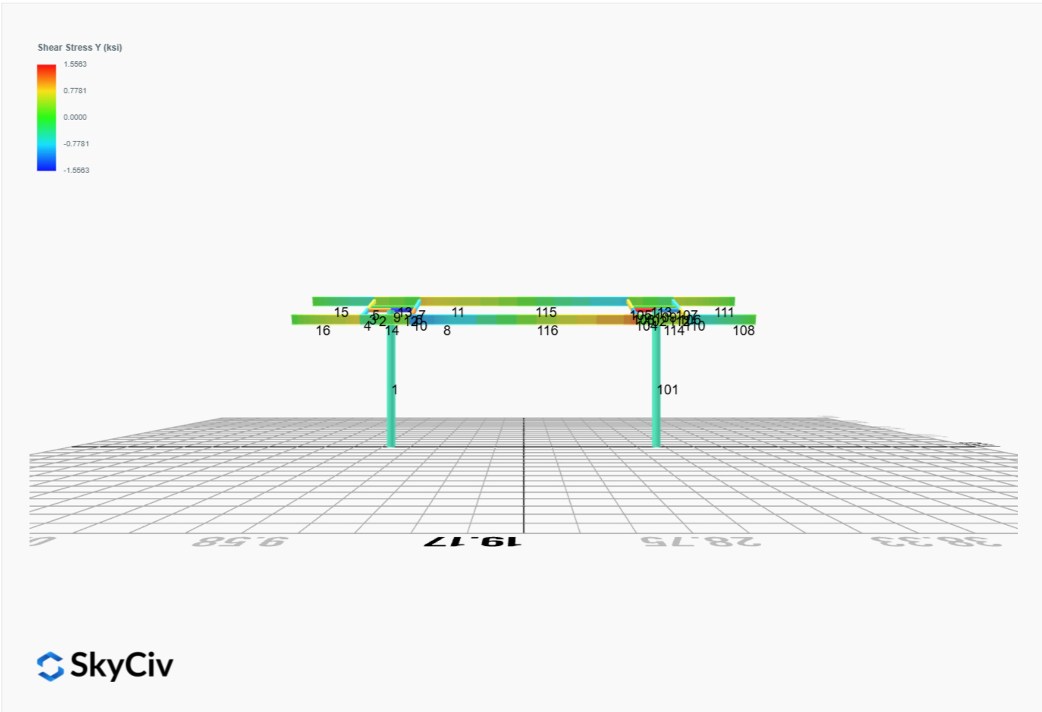

SkyCiv

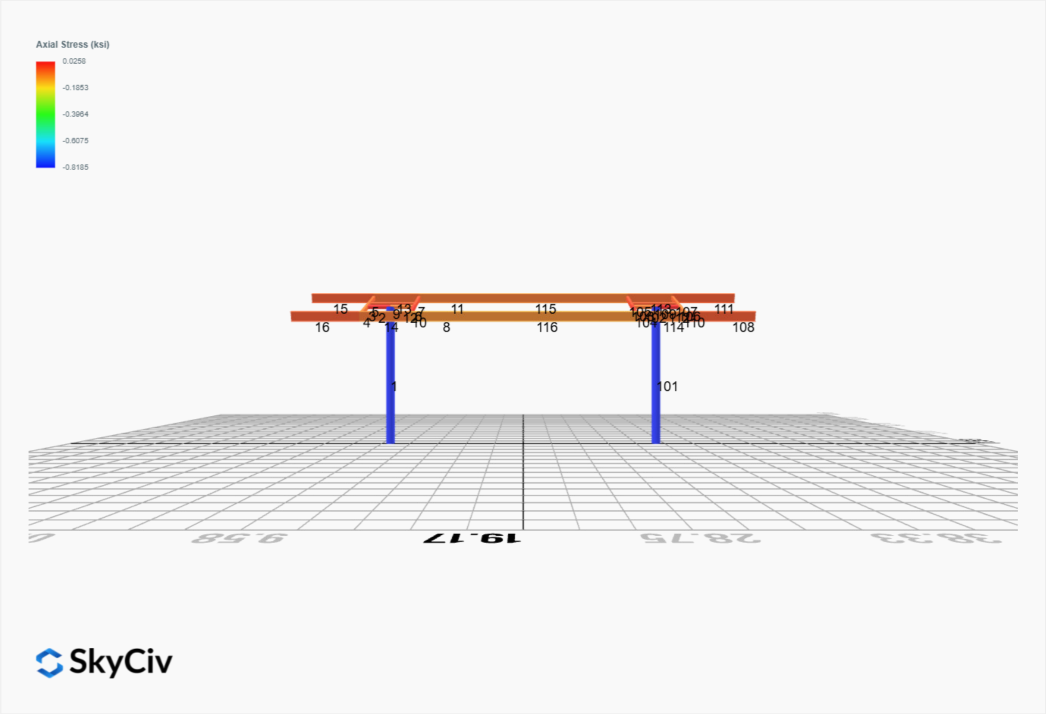
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
4.0783
2.6361
1.1938
-0.2484
-1.6906
15
11
115
111
16
4
14
10
8
116
104
114
110
108
1
101
SkyCiv


Shear Stress Y (ksi)
1.5563
0.7781
0.0000
-0.7781
-1.5563
15
11
115
111
16
4
14
10
8
116
104
114
110
108
1
101
SkyCiv


Axial Stress (ksi)
0.0258
-0.1853
-0.3964
-0.6075
-0.8185
15
13
11
115
111
16
4
14
10
8
116
114
110
108
1
101
19.17
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	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 2. D + L	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 3. D + (S or Lr or R)	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 3. D + (S or Lr or R)	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 5b. D + 0.7E	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	3.1840	0.1057	0.3669	-0.0460	0.0338
ULS: 8. 0.6D + 0.7E	0.0000	1.9104	0.0634	0.2202	-0.0276	0.0203
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4570	8.6216	0.3366	1.1578	-0.2831	19.6837
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4570	8.6216	0.3366	1.1578	-0.2831	19.6837
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1053	-0.9411	-0.0677	-0.2250	0.1328	-9.6908
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9546	-0.3785	-0.0461	-0.1514	0.1105	-24.2942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0928	7.2622	0.2789	0.9601	-0.2238	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0928	7.2622	0.2789	0.9601	-0.2238	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8290	0.0902	-0.0243	-0.0770	0.0881	-7.2596
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7159	0.5121	-0.0081	-0.0218	0.0714	-18.2122
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0928	7.2622	0.2789	0.9601	-0.2238	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0928	7.2622	0.2789	0.9601	-0.2238	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8290	0.0902	-0.0243	-0.0770	0.0881	-7.2596
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7159	0.5121	-0.0081	-0.0218	0.0714	-18.2122
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4570	7.3480	0.2943	1.0111	-0.2647	19.6702
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4570	7.3480	0.2943	1.0111	-0.2647	19.6702
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1053	-2.2147	-0.1100	-0.3718	0.1512	-9.7043
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9546	-1.6522	-0.0884	-0.2982	0.1289	-24.3077

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 (kip, kip-ft)
Axial	12.8835
Shear X	-2.4283
Shear Z	0.5125
Moment X	1.7636
Moment Y (Twist)	0.4504
Moment Z	41.1177

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	8.6216
Shear X	-1.4570
Shear Z	0.3366
Moment X	1.1578
Moment Y (Twist)	0.2831
Moment Z	24.3077

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	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 2. D + L	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 3. D + (S or Lr or R)	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 3. D + (S or Lr or R)	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 5b. D + 0.7E	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	3.1840	-0.1057	-0.3670	0.0460	0.0338
ULS: 8. 0.6D + 0.7E	-0.0000	1.9104	-0.0634	-0.2202	0.0276	0.0203
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4570	8.6216	-0.3366	-1.1579	0.2832	19.6838
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.4570	8.6216	-0.3366	-1.1579	0.2832	19.6838
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1053	-0.9411	0.0677	0.2250	-0.1328	-9.6907
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9546	-0.3785	0.0461	0.1514	-0.1105	-24.2942
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0928	7.2622	-0.2789	-0.9601	0.2239	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0928	7.2622	-0.2789	-0.9601	0.2239	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8290	0.0902	0.0243	0.0770	-0.0881	-7.2596
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7159	0.5121	0.0081	0.0218	-0.0714	-18.2122
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0928	7.2622	-0.2789	-0.9601	0.2239	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0928	7.2622	-0.2789	-0.9601	0.2239	14.7713
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8290	0.0902	0.0243	0.0770	-0.0881	-7.2596
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7159	0.5121	0.0081	0.0218	-0.0714	-18.2122
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4570	7.3480	-0.2943	-1.0111	0.2648	19.6702
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.4570	7.3480	-0.2943	-1.0111	0.2648	19.6702
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1053	-2.2147	0.1100	0.3717	-0.1512	-9.7043
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9546	-1.6522	0.0884	0.2982	-0.1289	-24.3077

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 (kip, kip-ft)
Axial	12.8835
Shear X	-2.4284
Shear Z	-0.5125
Moment X	-1.7636
Moment Y (Twist)	0.4507
Moment Z	41.1183

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	8.6216
Shear X	-1.4570
Shear Z	-0.3366
Moment X	-1.1579
Moment Y (Twist)	0.2832
Moment Z	24.3077

Project Details

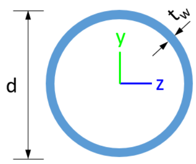
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Unit System:	imperial

Design Input Information

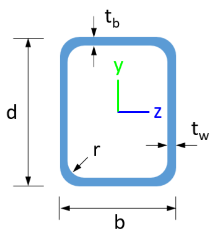
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

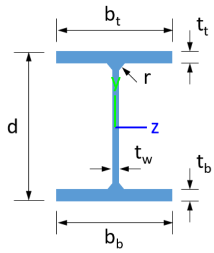
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)		
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23		



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)

3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	9	24.34	24.34	11.59	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.29,1.17,1.18,1.18,1.29,1.17,1.18,1.18,1.17,1.17	300	200	1
4	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.62,1.68,1.67,1.67,1.62,1.68,1.67,1.67,1.66,1.68	300	200	1
5	17	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.86,1.66,1.67,1.67,1.86,1.66,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.33,1.17,1.18,1.18,1.33,1.17,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.96,1.66,1.67,1.67,1.96,1.66,1.67,1.67,1.66,1.67	300	200	1
8	20	1.33	1.33	2.05	1.54,1.54,1.54,1.54,1.54,1.54,1.54,1.54,1.54,1.62,1.54,1.54,1.54,1.62,1.54,1.54,1.55,1.58,1.54,1.54,1.55,1.58,1.54,1.54,1.54,1.64	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.66,1.68	300	200	1
11	20	1.33	1.33	2.05	1.55,1.55,1.55,1.55,1.55,1.55,1.55,1.55,1.53,1.57,1.55,1.55,1.53,1.57,1.55,1.55,2.14,1.57,1.55,1.55,2.14,1.57,1.55,1.55,1.53,1.57	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.09,1.17,1.12,1.12,1.09,1.17,1.12,1.12,2.15,1.20,1.12,1.12,2.15,1.20,1.12,1.12,1.09,1.16	300	200	1
14	20	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.10,1.13,1.32,1.10,1.10,1.13,1.32,1.11,1.11,1.16,1.20,1.11,1.11,1.16,1.20,1.10,1.10,1.13,1.44	300	200	1
15	20	7.88	7.88	3.75	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	300	200	1
16	20	7.88	7.88	3.75	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	300	200	1
101	9	24.34	24.34	11.59	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.33,1.17,1.18,1.18,1.33,1.17,1.18,1.18,1.18,1.18	300	200	1
104	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.66,1.68	300	200	1
105	17	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.96,1.66,1.67,1.67,1.96,1.66,1.67,1.67,1.66,1.67	300	200	1
106	17	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.29,1.17,1.18,1.18,1.29,1.17,1.18,1.18,1.17,1.17	300	200	1
107	17	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.86,1.66,1.67,1.67,1.86,1.66,1.67,1.67,1.66,1.66	300	200	1
108	20	7.88	7.88	3.75	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	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.62,1.68,1.67,1.67,1.62,1.68,1.67,1.67,1.66,1.68	300	200	1
111	20	7.88	7.88	3.75	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	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1

113	20	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.09,1.17,1.12,1.12,1.09,1.17,1.12,1.12,2.15,1.20,1.12,1.12,2.15,1.20,1.12,1.12,1.09,1.16	300	200	1
114	20	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.10,1.13,1.32,1.10,1.10,1.13,1.32,1.11,1.11,1.16,1.20,1.11,1.11,1.16,1.20,1.10,1.10,1.13,1.44	300	200	1
115	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.15,1.11,1.11,1.11,1.15,1.11,1.11,1.11,1.11,1.11	300	200	1
116	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11	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	377.97	183.49	83.29	83.29	113.39	113.39
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	33.21	6.46	56.26	44.91
14	159.30	97.43	33.74	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	377.97	183.49	83.29	83.29	113.39	113.39
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	55.15	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	55.15	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	33.21	6.46	56.26	44.91
114	159.30	97.43	33.74	6.46	56.26	44.91
115	159.30	48.27	14.70	6.46	56.26	44.91
116	159.30	48.27	14.71	6.46	56.26	44.91

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.070	0.494	0.050	0.021	0.005	0.502	#32	0.497	Not Required	Pass
2	0.002	0.321	0.078	0.072	0.015	0.400	#13	0.036	Not Required	Pass
3	0.002	0.553	0.011	0.054	0.004	0.565	#13	0.046	Not Required	Pass
4	0.001	0.467	0.038	0.047	0.009	0.504	#13	0.082	Not Required	Pass

5	0.001	0.343	0.021	0.055	0.006	0.347	#13	0.076	Not Required	Pass
6	0.002	0.700	0.028	0.071	0.006	0.728	#13	0.046	Not Required	Pass
7	0.002	0.433	0.042	0.069	0.012	0.449	#13	0.076	Not Required	Pass
8	0.002	0.092	0.050	0.041	0.004	0.095	#13	0.102	Not Required	Pass
9	0.003	0.083	0.037	0.003	0.002	0.121	#13	0.206	Not Required	Pass
10	0.003	0.603	0.039	0.060	0.009	0.609	#13	0.082	Not Required	Pass
11	0.002	0.106	0.051	0.047	0.004	0.108	#13	0.102	Not Required	Pass
12	0.001	0.474	0.095	0.094	0.017	0.570	#13	0.054	Not Required	Pass
13	0.003	0.187	0.108	0.059	0.005	0.241	#13	0.306	Not Required	Pass
14	0.003	0.163	0.106	0.050	0.005	0.215	#13	0.204	Not Required	Pass
15	0.000	0.053	0.027	0.024	0.002	0.077	#13	Not Required	Not Required	Pass
16	0.000	0.046	0.027	0.020	0.002	0.069	#13	Not Required	Not Required	Pass
101	0.070	0.494	0.050	0.021	0.005	0.502	#32	0.497	Not Required	Pass
102	0.001	0.474	0.095	0.094	0.017	0.570	#13	0.054	Not Required	Pass
103	0.002	0.700	0.028	0.071	0.006	0.728	#13	0.046	Not Required	Pass
104	0.003	0.603	0.039	0.060	0.009	0.609	#13	0.082	Not Required	Pass
105	0.002	0.433	0.042	0.069	0.012	0.449	#13	0.076	Not Required	Pass
106	0.002	0.553	0.011	0.054	0.004	0.565	#13	0.046	Not Required	Pass
107	0.001	0.343	0.021	0.055	0.006	0.347	#13	0.076	Not Required	Pass
108	0.000	0.046	0.027	0.020	0.002	0.069	#13	Not Required	Not Required	Pass
109	0.003	0.083	0.037	0.003	0.002	0.121	#13	0.206	Not Required	Pass
110	0.001	0.467	0.038	0.047	0.009	0.504	#13	0.082	Not Required	Pass
111	0.000	0.053	0.027	0.024	0.002	0.077	#13	Not Required	Not Required	Pass
112	0.002	0.321	0.078	0.072	0.015	0.400	#13	0.036	Not Required	Pass
113	0.003	0.187	0.108	0.059	0.005	0.241	#13	0.204	Not Required	Pass
114	0.003	0.163	0.106	0.050	0.005	0.215	#13	0.306	Not Required	Pass
115	0.006	0.692	0.057	0.047	0.004	0.743	#13	0.644	Not Required	Pass
116	0.005	0.597	0.058	0.041	0.004	0.651	#13	0.644	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

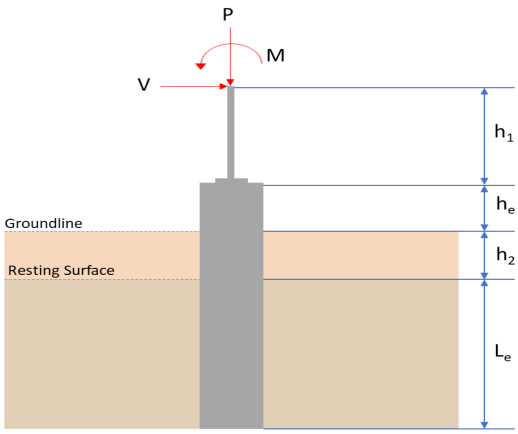
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 6.5\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - 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)	8.622	12.884
V_x (kip)	-1.457	-2.428
V_z (kip)	0.337	0.512
M_x (kipft)	1.158	1.764
M_z (kipft)	24.308	41.118

Material Properties

$f'_{ck} = 2.5\ \text{ksi}$ - 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,

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

$$H_o = \frac{(-1.457\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.23201\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.308 \text{ kipft}) + ((-1.457 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8707 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.0821 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.337 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.053662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.158 \text{ kipft}) + ((0.337 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.18439 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.8863 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.0821 \text{ ft}), (2.8863 \text{ ft})]$$

$$L_{e,req} = 6.082 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.082 \text{ ft})}{(6.5 \text{ ft})}$$

$$Ratio = 0.93569$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.622\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.53888\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.53888\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.26944$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.23201\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.8707\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8707\ \text{kipft/ft}) \times (6.5\ \text{ft})) + \left(3 \times (-0.23201\ \text{kip/ft}) \times (6.5\ \text{ft})^2\right)}{(6 \times (3.8707\ \text{kipft/ft})) + (4 \times (-0.23201\ \text{kip/ft}) \times (6.5\ \text{ft}))}$$

$$a = 4.445\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8707\ \text{kipft/ft})) + (3 \times (-0.23201\ \text{kip/ft}) \times (6.5\ \text{ft}))]^2}{(6.5\ \text{ft})^2 \times [(3 \times (3.8707\ \text{kipft/ft})) + (2 \times (-0.23201\ \text{kip/ft}) \times (6.5\ \text{ft}))]}$$

$$p = 0.248\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.8707\ \text{kipft/ft})) + ((-0.23201\ \text{kip/ft}) \times (6.5\ \text{ft}))]}{(6.5\ \text{ft})^2}$$

$$s = 0.88521\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.445\ \text{ft})}{2}$$

$$p_a = 0.33338\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

	$$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(0.248\ \mathrm{kip/ft^2}\right)}{\left(0.33338\ \mathrm{kip/ft^2}\right)}$$ $$Ratio = 0.74391$$	Status: **PASS** Ratio: **0.740**
	p_s - Allowable lateral soil pressure at depth L_e, $$p_s = R\, L_e$$ $$p_s = (150\ \mathrm{psf/ft}) \times (6.5\ \mathrm{ft})$$ $$p_s = 0.975\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$ $$Ratio = \frac{\left(0.88521\ \mathrm{kip/ft^2}\right)}{\left(0.975\ \mathrm{kip/ft^2}\right)}$$ $$Ratio = 0.90791$$	Status: **PASS** Ratio: **0.910**
	Considering z-direction: $H_o = 0.053662\ \mathrm{kip/ft}$ - Lateral force per length of pile, $M_o = 0.18439\ \mathrm{kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $$a = \frac{(4\, M_o\, L_e) + \left(3\, H_o\, L_e^2\right)}{(6\, M_o) + (4\, H_o\, L_e)}$$ $$a = \frac{(4 \times (0.18439\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (0.18439\ \mathrm{kipft/ft})) + (4 \times (0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$ $$a = 4.6354\ \mathrm{ft}$$ p - Earth pressure against the pile at distance a/2 from resting surface, $$p = \frac{0.75\, [(4\, M_o) + (3\, H_o\, L_e)]^2}{L_e^2\, [(3\, M_o) + (2\, H_o\, L_e)]}$$ $$p = \frac{0.75 \times [(4 \times (0.18439\ \mathrm{kipft/ft})) + (3 \times (0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^{\ 2}}{(6.5\ \mathrm{ft})^2 \times [(3 \times (0.18439\ \mathrm{kipft/ft})) + (2 \times (0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$ $$p = 0.045169\ \mathrm{kip/ft^2}$$ s - Earth pressure against the pile at distance L_e, $$s = \frac{6\, [(2\, M_o) + (H_o\, L_e)]}{L_e^2}$$ $$s = \frac{6 \times [(2 \times (0.18439\ \mathrm{kipft/ft})) + ((0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$ $$s = 0.10191\ \mathrm{kip/ft^2}$$ **Check lateral soil pressure capacity:** p_a - Allowable lateral soil pressure at depth a/2, $$p_a = R\, \frac{a}{2}$$ $$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6354\ \mathrm{ft})}{2}$$ $$p_a = 0.34766\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(0.045169\ \mathrm{kip/ft^2}\right)}{\left(0.34766\ \mathrm{kip/ft^2}\right)}$$	

$$Ratio = 0.12992$$

Status: **PASS**
Ratio: **0.130**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

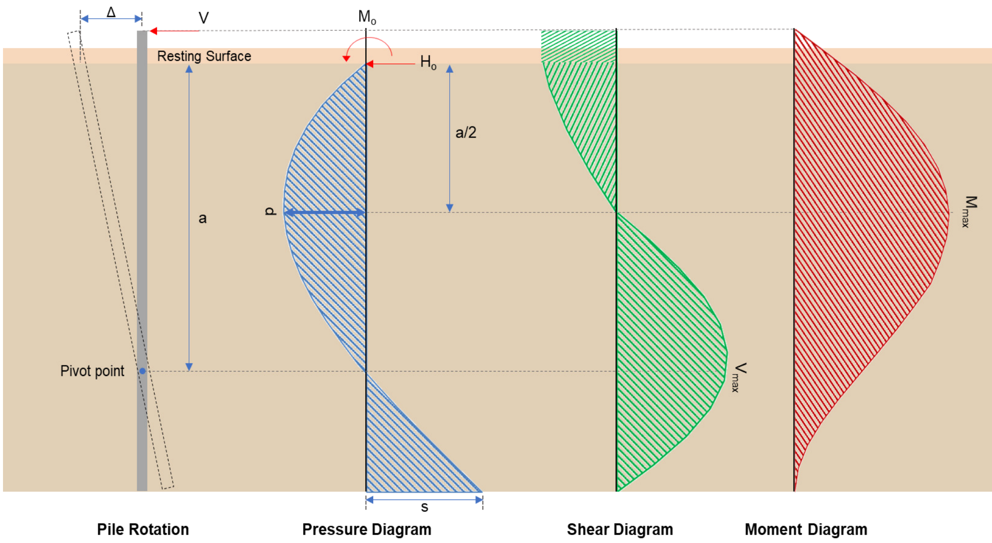
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.10191 \text{ kip/ft}^2\right)}{\left(0.975 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.10452$$

Status: **PASS**
Ratio: **0.100**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.428 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.38662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.118 \text{ kipft}) + ((-2.428 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5475 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5475 \text{ kipft/ft})}{(-0.38662 \text{ kip/ft})}$$

$$E = 16.935 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5475 \text{ kipft/ft}) \times (6.5 \text{ ft})) + \left(3 \times (-0.38662 \text{ kip/ft}) \times (6.5 \text{ ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (6.5475\ \text{kipft/ft})) + (4 \times (-0.38662\ \text{kip/ft}) \times (6.5\ \text{ft}))}$$

$$a = 4.4437\ \text{ft}$$

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

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

$$V_{max} = ((-0.38662\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.935\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.4437\ \text{ft})}{(6.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.935\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.4437\ \text{ft})}{(6.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.1544\ \text{kip}$$

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

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

$$M_{max} = ((-0.38662\ \text{kip/ft}) \times (48\ \text{in}) \times (6.5\ \text{ft})) \times \left[\left(\frac{(16.935\ \text{ft})}{(6.5\ \text{ft})} + \frac{(4.4437\ \text{ft})}{2 \times (6.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (16.935\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.4437\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.935\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.4437\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 25.585\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.512\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.081529\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.764\ \text{kipft}) + ((0.512\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.28089\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.28089\ \text{kipft/ft})}{(0.081529\ \text{kip/ft})}$$

$$E = 3.4453\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28089\ \text{kipft/ft}) \times (6.5\ \text{ft})) + \left(3 \times (0.081529\ \text{kip/ft}) \times (6.5\ \text{ft})^2\right)}{(6 \times (0.28089\ \text{kipft/ft})) + (4 \times (0.081529\ \text{kip/ft}) \times (6.5\ \text{ft}))}$$

$$a = 4.6351\ \text{ft}$$

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

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

$$V_{max} = ((0.081529\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.6351\ \text{ft})}{(6.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.6351\ \text{ft})}{(6.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.52296\ \text{kip}$$

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

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

$$M_{max} = ((0.081529\ \text{kip/ft}) \times (48\ \text{in}) \times (6.5\ \text{ft})) \times \left[\left(\frac{(3.4453\ \text{ft})}{(6.5\ \text{ft})} + \frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.5104\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5\ \text{ksi}$ - Concrete strength,
$f_{yk} = 60\ \text{ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(12.884\ \text{kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.168\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.168\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\, \frac{\pi\, d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625\ \text{in})^2}{4}$$

$$A_{st} = 4.2951\ \text{in}^2$$

$Ratio$ - Capacity

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

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

	$$Ratio = 0.96556$$	Status: **PASS** Ratio: **0.970**
25.2.3	s_{rebar} - Minimum spacing of reinforcement, $$s_{rebar} = Max\left[1.5, \left(1.5\ d_{bar}\right)\right]$$ $$s_{rebar} = Max\left[1.5, \left(1.5 \times \left(0.625\ \text{in}\right)\right)\right]$$ $$s_{rebar} = 1.5\ \text{in}$$ **Ties:**	
25.7.2.2 *25.7.2.1*	Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) s_{ties} - Maximum spacing of ties, $$s_{ties} = Min\left[\left(16\ d_{bar}\right), \left(48\ d_{ties}\right), Min\left(D, b\right)\right]$$ $$s_{ties} = Min\left[\left(16 \times \left(0.625\ \text{in}\right)\right), \left(48 \times \left(0.375\ \text{in}\right)\right), Min\left(\left(48\ \text{in}\right), \left(48\ \text{in}\right)\right)\right]$$ $$s_{ties} = 10\ \text{in}$$ **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 $$\phi P_N = \phi\ 0.80\ \left[\left(0.85\ f'_{ck}\ \left[A_g - A_{st}\right]\right) + \left(f_{yk}\ A_{st}\right)\right]$$ $$\phi P_N = \left(0.65\right) \times 0.80 \times \left[\left(0.85 \times \left(2.5\ \text{ksi}\right) \times \left[\left(2304\ \text{in}^2\right) - \left(4.2951\ \text{in}^2\right)\right]\right) + \left(\left(60\ \text{ksi}\right) \times \left(4.2951\ \text{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \text{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{\left(12.884\ \text{kip}\right)}{\left(2675.2\ \text{kip}\right)}$$ $$Ratio = 0.0048161$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 *22.5.5.1.3* *22.5.5.1.1*	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\ \text{in}$ - Effective width, d - Effective depth $$d = 0.80\ D$$ $$d = 0.80 \times \left(48\ \text{in}\right)$$ $$d = 38.4\ \text{in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{\left(38.4\ \text{in}\right)}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\ \lambda_s\ \sqrt{f'_{ck}}\ b_w\ d$$ $$V_{c,max} = 5 \times \left(0.64282\right) \times \sqrt{\left(2500\ \text{psi}\right)} \times \left(48\ \text{in}\right) \times \left(38.4\ \text{in}\right)$$	

$$V_{c,max} = 296.21 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.884 \text{ kip} \rightarrow 12884 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \frac{P}{6\,A_g}\right]\,b_w\,d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12884 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.2 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

$$V_{c,b} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \left(0.05\,f'_{ck}\right)\right]\,b_w\,d$$

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

$$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$

$$V_c = Min\left[(296.21 \text{ kip}), (120.2 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.2 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\,\left(V_c + V_s\right)$$

$$\phi V_n = (0.65) \times ((120.2 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.21 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.1544 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(8.1544\ \text{kip})}{(111.21\ \text{kip})}$$

$$Ratio = 0.073322$$

Status: **PASS**
Ratio: **0.070**

Considering z-direction:

$V_{max} = 0.52296\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.52296\ \text{kip})}{(111.21\ \text{kip})}$$

$$Ratio = 0.0047023$$

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:
$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 ksi)} \times 18432.001 in^3$$

$$\phi M_{n,1} = 249.600 kipft$$

14.5.2.1b $\phi M_{n,2}$

$$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \text{ksi}) \times \left(18432\ \text{in}^3\right)$$

$$\phi M_{n,2} = 2121.6\ \text{kipft}$$

Therefore,
ϕM_n - Allowable flexural strength,

$$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$

$$\phi M_n = MIN[(249.6\ \text{kipft}), (2121.6\ \text{kipft})]$$

$$\phi M_n = 249.6\ \text{kipft}$$

Considering x-direction:

$M_{max} = 25.585\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

$$Ratio = \frac{(25.585\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.1025$$

Status: **PASS**
Ratio: **0.100**

Considering z-direction:

$M_{max} = 1.5104\ \text{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

$$Ratio = \frac{M_{max}}{\phi M_n}$$

	$$Ratio = \frac{(1.5104\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.0060513$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

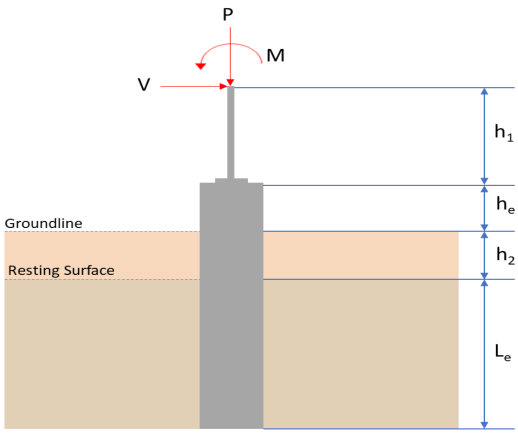
SkyCiv Foundation Design

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
$L = 6.5\ \mathrm{ft}$ - 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)	8.622	12.884
V_x (kip)	-1.457	-2.428
V_z (kip)	-0.337	-0.512
M_x (kipft)	-1.158	-1.764
M_z (kipft)	24.308	41.118

Material Properties

$f'_{ck} = 2.5\ \mathrm{ksi}$ - 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,

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

$$H_o = \frac{(-1.457\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.23201\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.308\ \text{kipft}) + ((-1.457\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.8707\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.0821\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.337\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.053662\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.158\ \text{kipft}) + ((-0.337\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.18439\ \text{kipft/ft}$$

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:
$L_{e,z} = 2.0203\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.0821\ \text{ft}), (2.0203\ \text{ft})]$$

$$L_{e,req} = 6.082\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.5\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.082\ \text{ft})}{(6.5\ \text{ft})}$$

$$Ratio = 0.93569$$

Status: **PASS**
Ratio: **0.940**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\ D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.622\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.53888\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.53888\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.26944$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.23201\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.8707\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8707\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.23201\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (3.8707\ \mathrm{kipft/ft})) + (4 \times (-0.23201\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.445\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.8707\ \mathrm{kipft/ft})) + (3 \times (-0.23201\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^{\,2}}{(6.5\ \mathrm{ft})^2 \times [(3 \times (3.8707\ \mathrm{kipft/ft})) + (2 \times (-0.23201\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$

$$p = 0.248\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (3.8707\ \mathrm{kipft/ft})) + ((-0.23201\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$

$$s = 0.88521\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.445\ \mathrm{ft})}{2}$$

$$p_a = 0.33338\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.248\ \mathrm{kip/ft^2}\right)}{\left(0.33338\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.74391$$

Status: **PASS**
Ratio: **0.740**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.5\ \mathrm{ft})$$

$$p_s = 0.975\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.88521\ \mathrm{kip/ft^2}\right)}{\left(0.975\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.90791$$

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = -0.053662\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.18439\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18439\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (0.18439\ \mathrm{kipft/ft})) + (4 \times (-0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.6354\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.18439\ \mathrm{kipft/ft})) + (3 \times (-0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]^2}{(6.5\ \mathrm{ft})^2 \times [(3 \times (0.18439\ \mathrm{kipft/ft})) + (2 \times (-0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}$$

$$p = -0.011723\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.18439\ \mathrm{kipft/ft})) + ((-0.053662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))]}{(6.5\ \mathrm{ft})^2}$$

$$s = 0.002838\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.6354\ \mathrm{ft})}{2}$$

$$p_a = 0.34766\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.011723\ \mathrm{kip/ft^2}\right)}{\left(0.34766\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.033721$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.5\ \mathrm{ft})$$

$$p_s = 0.975\ \mathrm{kip/ft^2}$$

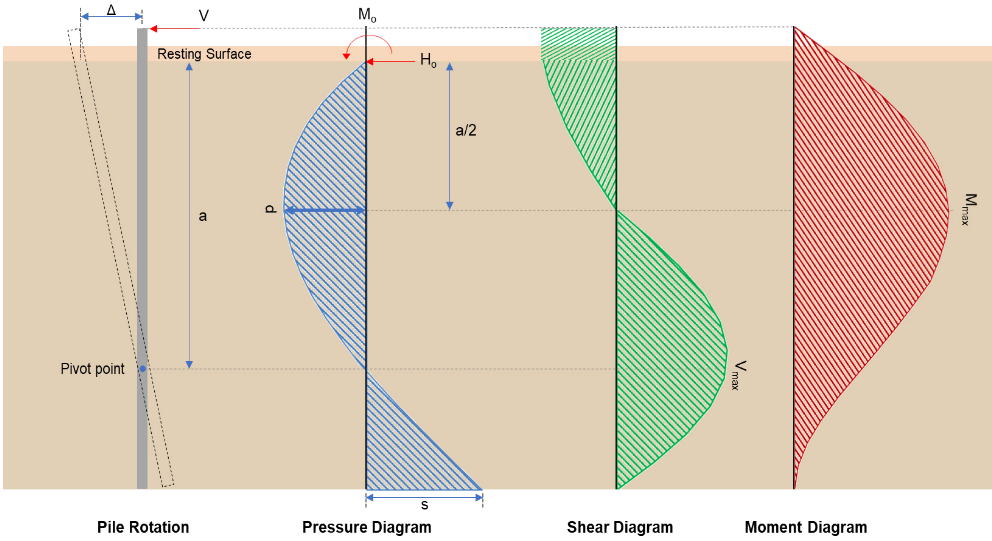
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.002838\ \mathrm{kip/ft^2})}{(0.975\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.0029107$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.428\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.38662\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(41.118\ \mathrm{kipft}) + ((-2.428\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 6.5475\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5475\ \mathrm{kipft/ft})}{(-0.38662\ \mathrm{kip/ft})}$$

$$E = 16.935\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5475\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.38662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (6.5475\ \mathrm{kipft/ft})) + (4 \times (-0.38662\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.4437\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.38662\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.935\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.4437\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.935\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.4437\ \mathrm{ft})}{(6.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 8.1544\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.38662\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.5\ \mathrm{ft})) \times \left[\left(\frac{(16.935\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + \frac{(4.4437\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (16.935\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.4437\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.935\ \mathrm{ft})}{(6.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.4437\ \mathrm{ft})}{2 \times (6.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 25.585\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.512\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.081529\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.764\ \mathrm{kipft}) + ((-0.512\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.28089\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.28089\ \mathrm{kipft/ft})}{(-0.081529\ \mathrm{kip/ft})}$$

$$E = 3.4453\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28089\ \mathrm{kipft/ft}) \times (6.5\ \mathrm{ft})) + \left(3 \times (-0.081529\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft})^2\right)}{(6 \times (0.28089\ \mathrm{kipft/ft})) + (4 \times (-0.081529\ \mathrm{kip/ft}) \times (6.5\ \mathrm{ft}))}$$

$$a = 4.6351\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.081529\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.6351\ \text{ft})}{(6.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.6351\ \text{ft})}{(6.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.52296\ \text{kip}$$

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

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

$$M_{max} = ((-0.081529\ \text{kip/ft}) \times (48\ \text{in}) \times (6.5\ \text{ft})) \times \left[\left(\frac{(3.4453\ \text{ft})}{(6.5\ \text{ft})} + \frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 3\right) \times \left(\frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.4453\ \text{ft})}{(6.5\ \text{ft})} + 2\right) \times \left(\frac{(4.6351\ \text{ft})}{2 \times (6.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.5104\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5\ \text{ksi}$ - Concrete strength,
$f_{yk} = 60\ \text{ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(12.884\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.168\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.168\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar}\, \frac{\pi\, d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625\ \text{in})^2}{4}$$

$$A_{st} = 4.2951\ \text{in}^2$$

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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,

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

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

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

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

$$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$

$$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times \left(2.5 \text{ ksi}\right) \times \left[\left(2304 \text{ in}^2\right) - \left(4.2951 \text{ in}^2\right)\right]\right) + \left(\left(60 \text{ ksi}\right) \times \left(4.2951 \text{ in}^2\right)\right)\right]$$

$$\phi P_N = 2675.2 \text{ kip}$$

$Ratio$ - Capacity

$$Ratio = \frac{P}{\phi P_N}$$

$$Ratio = \frac{\left(12.884 \text{ kip}\right)}{\left(2675.2 \text{ kip}\right)}$$

$$Ratio = 0.0048161$$

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

$$d = 0.80\, D$$

$$d = 0.80 \times \left(48 \text{ in}\right)$$

$$d = 38.4 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$

$$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{\left(38.4 \text{ in}\right)}{10}}}, 1\right]$$

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

$$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$

$$V_{c,max} = 5 \times \left(0.64282\right) \times \sqrt{\left(2500 \text{ psi}\right)} \times \left(48 \text{ in}\right) \times \left(38.4 \text{ in}\right)$$

$$V_{c,max} = 296.21 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.884 \text{ kip} \rightarrow 12884 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

$$V_{c,a} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \frac{P}{6\,A_g}\right]\,b_w\,d$$

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12884 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.2 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

$$V_{c,b} = \left[2\,\lambda_s\,\sqrt{f'_{ck}} + \left(0.05\,f'_{ck}\right)\right]\,b_w\,d$$

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi}))\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 348.89 \text{ kip}$$

V_c - Governing shear strength of concrete

$$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$

$$V_c = Min\left[(296.21 \text{ kip}), (120.2 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.2 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

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

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

$$V_s = MIN\left[(737.28 \text{ kip}), (50.894 \text{ kip})\right]$$

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

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi\,\left(V_c + V_s\right)$$

$$\phi V_n = (0.65) \times ((120.2 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.21 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.1544 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(8.1544 \text{ kip})}{(111.21 \text{ kip})}$$ $$Ratio = 0.073322$$	Status: **PASS** Ratio: **0.070**
	Considering z-direction: $V_{max} = 0.52296 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.52296 \text{ kip})}{(111.21 \text{ kip})}$$ $$Ratio = 0.0047023$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$ $$S_m = 18432 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001 in^3$$ $$\phi M_{n,1} = 249.600 kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 25.585 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(25.585 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.1025$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $M_{max} = 1.5104 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(1.5104\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.0060513$$	Status: **PASS** Ratio: **0.010**