Your Project Calculations

Project Name: CharkowskiOffGridCabin Exp C5 CU

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=CharkowskiOffGridCabin%20Exp%20C5%20CU&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=0Eu7vUhXuMHervG8Lbkzp1YuTK20ewHxVhTaMluIBwDAxH4fFCOEp58ziqCugk8l

Array Specification

Product:	Beam
Unique ID:	2P-15-6TOP-SD-12-L-4Hx4W-G8G2
Duty Classification:	SD
Module Width:	41.10 in
Module Length:	74.00in
Number of Rows:	4
Number of Columns:	4
Total Number of Modules:	16
Desired Tilt Angle:	60
Front Edge Clearance:	1.5
Total Array Height at Tilt:	13.44 ft
Total Frame Length:	24.50 ft
Frame Weight:	1172 lbs
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	25.00 ft
Rail Length:	166.40 in
Rail Spacing:	3.13 ft
Rail Check:	PASS (57% utilized)

Support Specifications

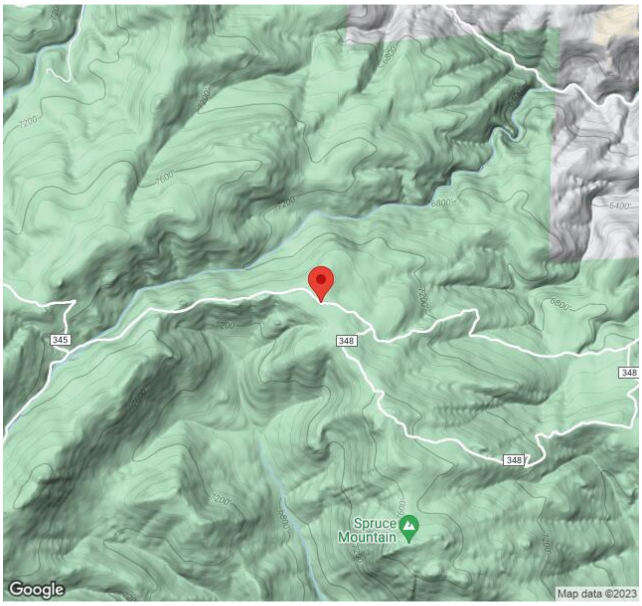
Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	7.50 ft
Number of Poles:	2
Pole Spacing:	15 ft

Foundation Specifications

Foundation Type:	Isolated Footing
Foundation Dimensions:	81 x 81 in
Foundation Depth (below grade):	30 in
Foundation Volume:	8.437 y^3
Foundation Result:	PASSED
Mount Twist:	1.757790 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	5431 Storm Mountain Rd, Loveland, CO 80538, USA
Wind Speed:	167 mph
Snow Load:	100 psf
Design Uplift Pressure:	0.047669 ksf
Design Downforce Pressure:	-0.047669 ksf
Design Snow Pressure:	0.010996 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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{"wind_speed_override":167,"snow_load_override":100,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","project_id":"CharkowskiOffGridCabin Exp C5 CU","site_address":"5431 Storm Mountain Rd, Loveland, CO 80538, USA","module_width":41.1,"module_length":74,"number_rows":"4","number_columns":4,"pole_mount_section":"4_40","core_pipe_width":65,"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":"2_12GA","pole_spacing":"15","tilt_angle":60,"ground_clearance":1.5,"risk_category":"I","exposure_category":"C","frame_duty_override":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Isolated","foundation_size":81,"isolated_foundation_thickness":30,"check_rails":true}
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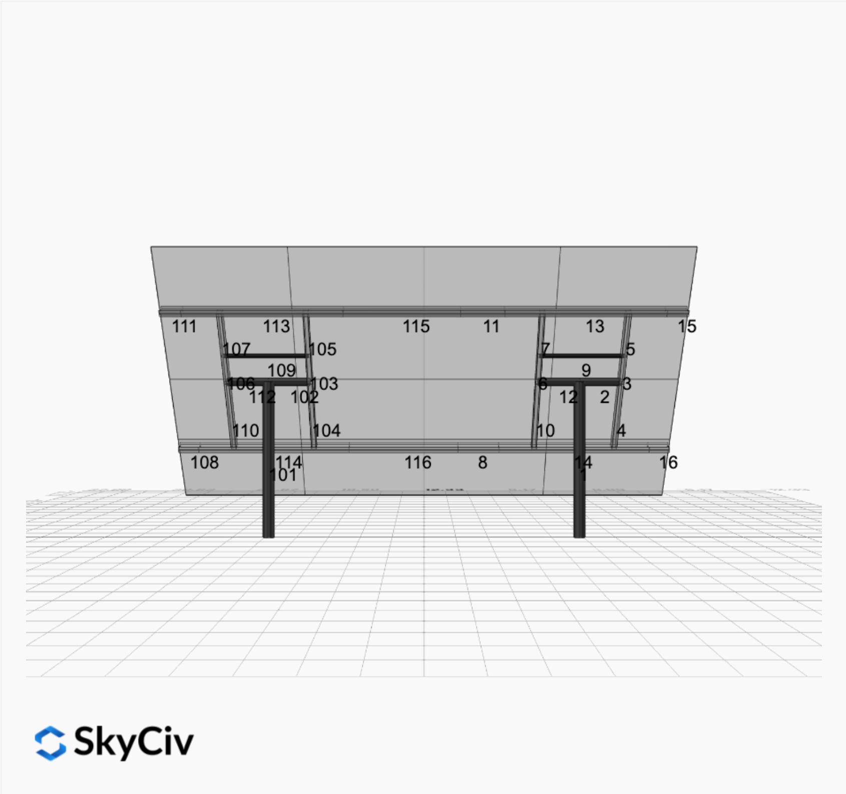
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles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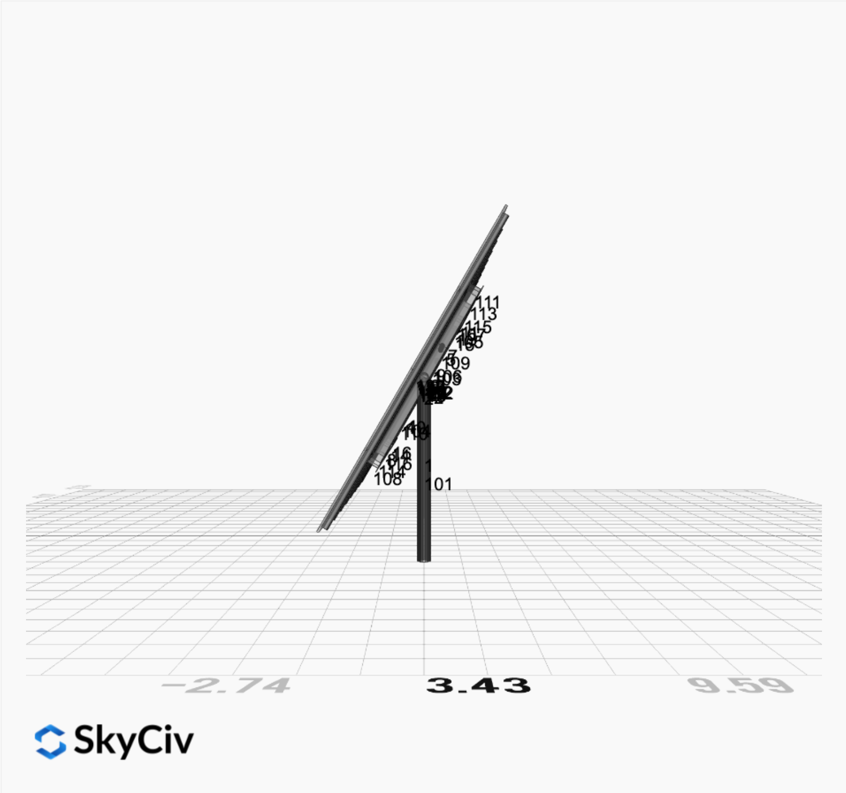
SkyCiv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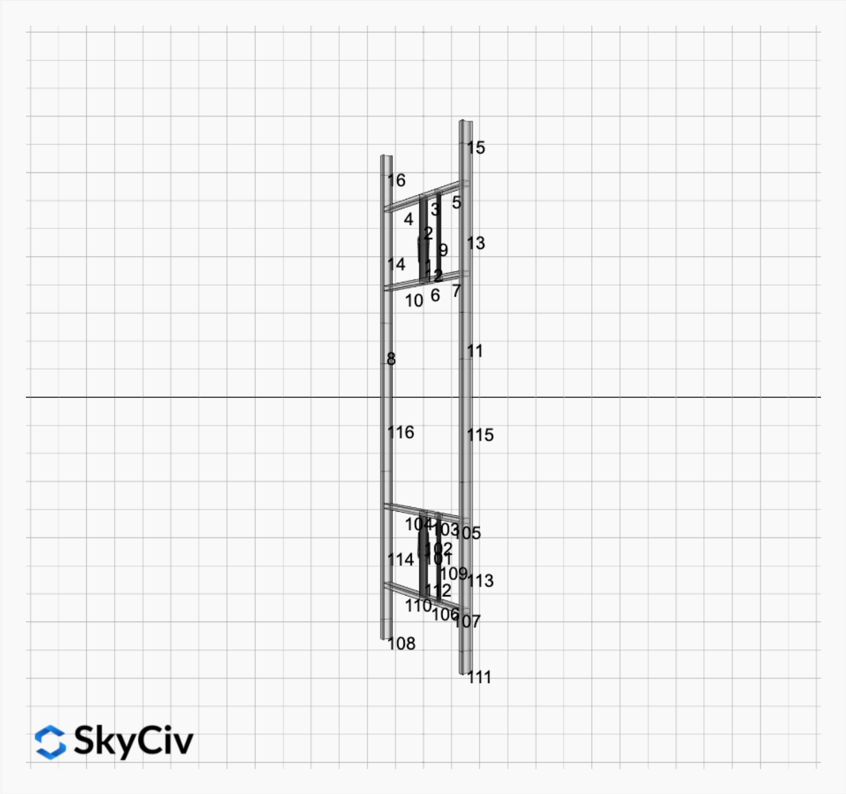
SkyCiv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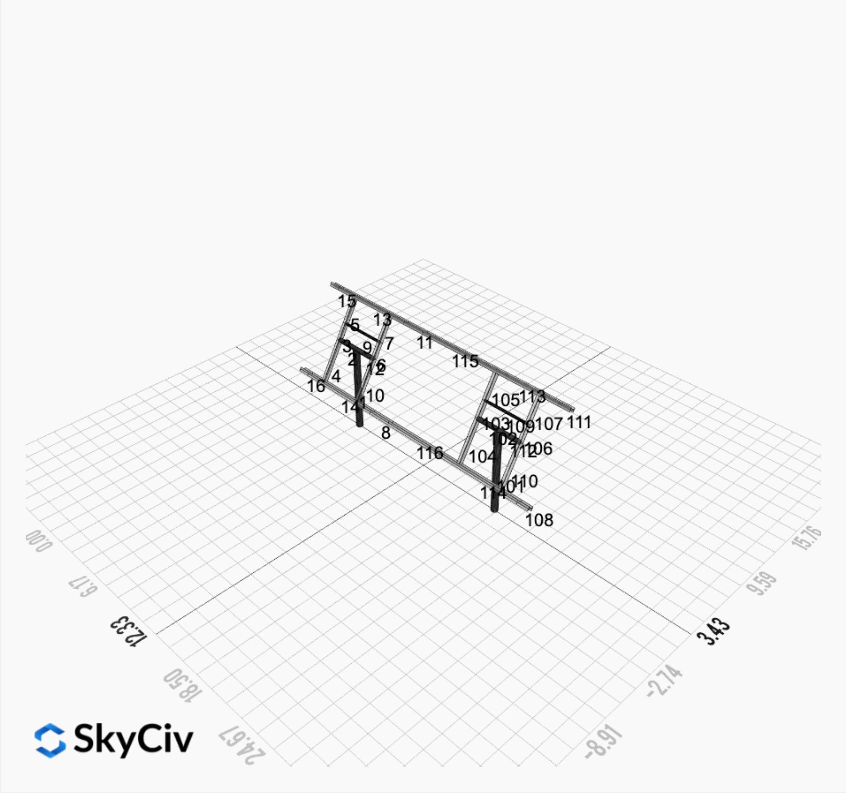
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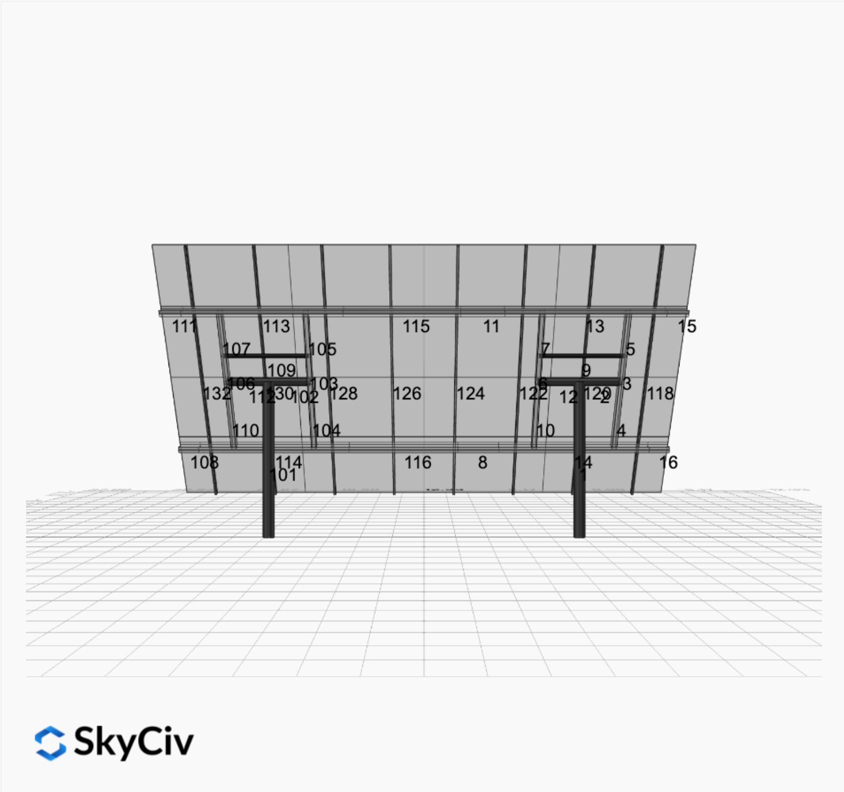
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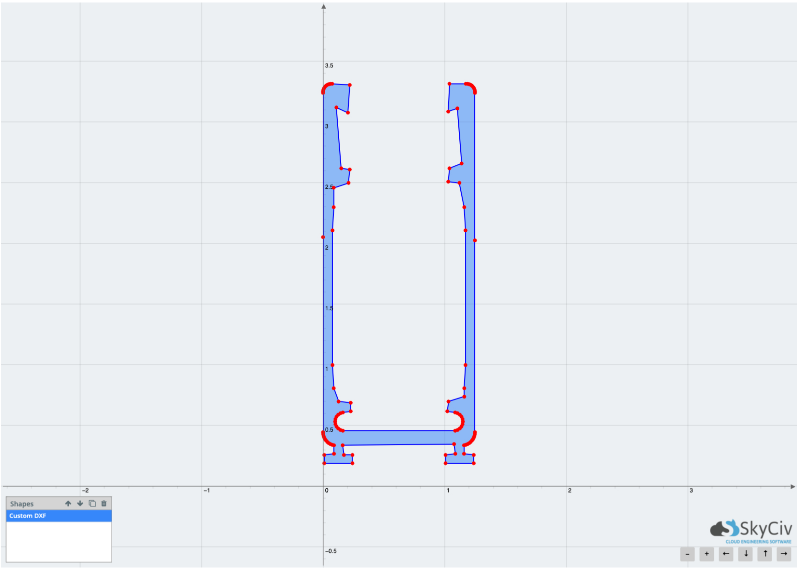
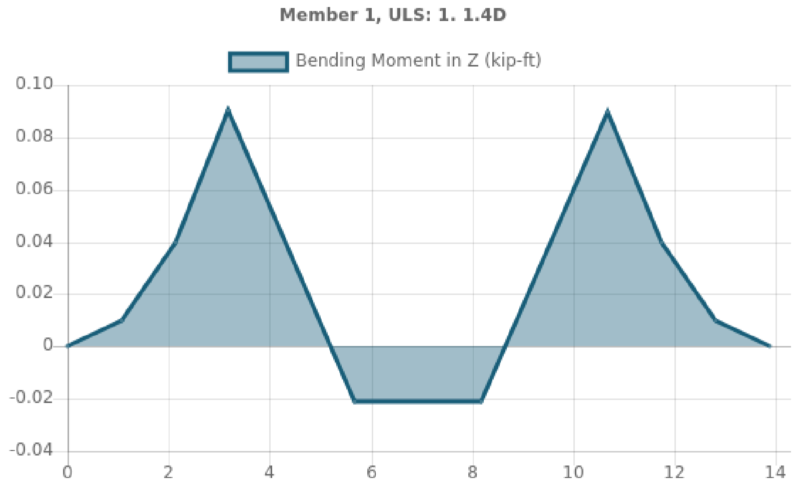
124
122
12
120
2
118
110
104
10
4
108
114
101
116
8
14
1
16
SkyCiv

Rail Design Check

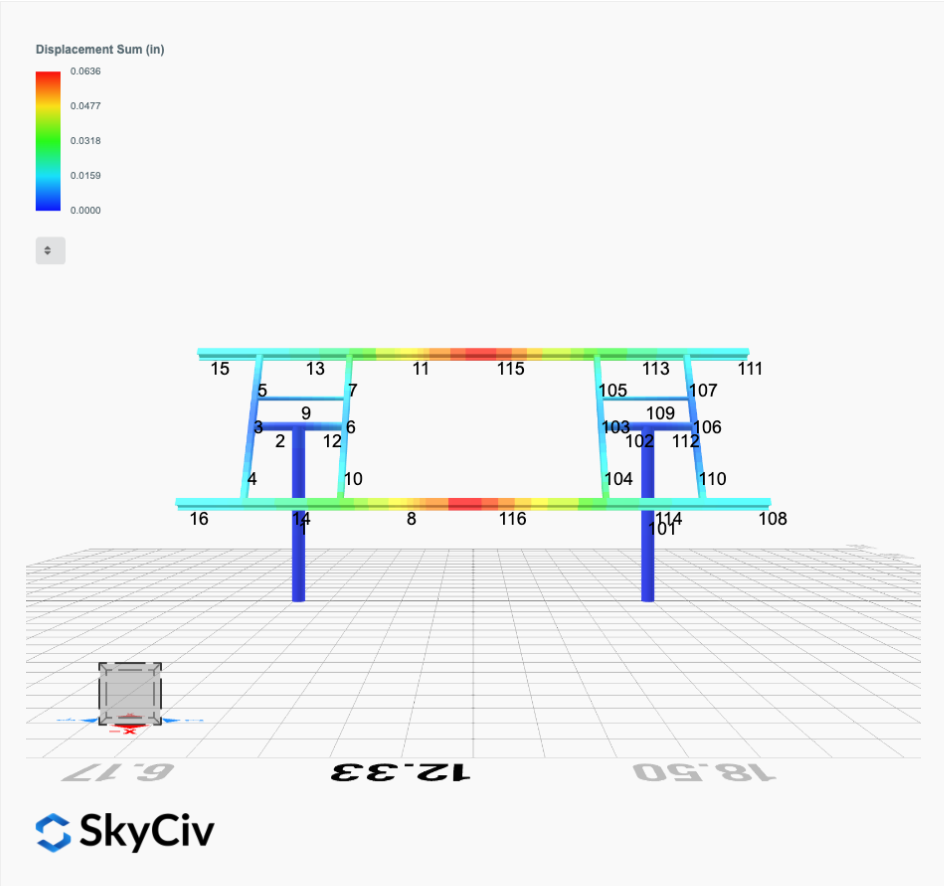
Rail Length: 13.866666666666667 ft
Additional Restraints Required: None
Tributary Width: 3.125 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0172 kip/ft
Snow (Y): -0.0298 kip/ft
Wind uplift Case A: 0.1490 kip/ft
Wind downforce Case A: 0.1490 kip/ft
Dead (Panel load) (X): 0.0073 kip/ft
Dead (Panel load) (Y): -0.0127 kip/ft

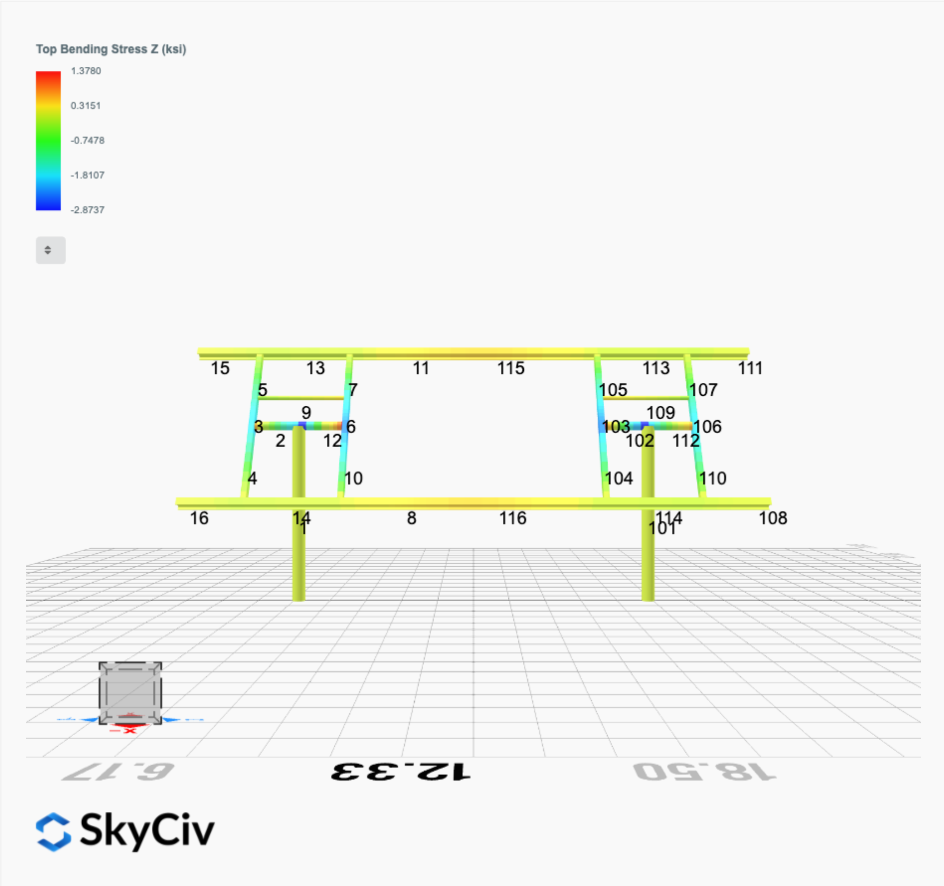


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	19.64179707	0.569	PASS
Material Yield	34.5	19.64179707	0.569	PASS
Material Strength	37	19.64179707	0.531	PASS



FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
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-1.8107
-2.8737
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13
11
115
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7
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107
9
109
3
6
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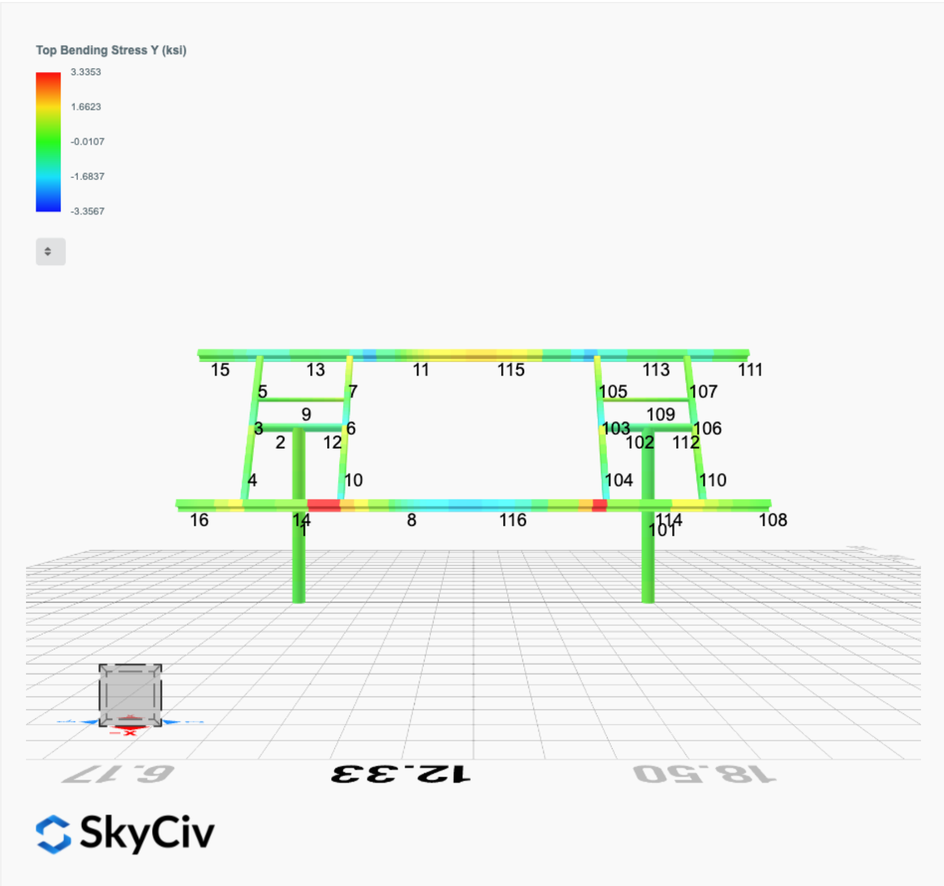
SkyCiv


Top Bending Stress Y (ksi)
3.3353
1.6623
-0.0107
-1.6837
-3.3567
15
13
11
115
113
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5
7
105
107
9
109
3
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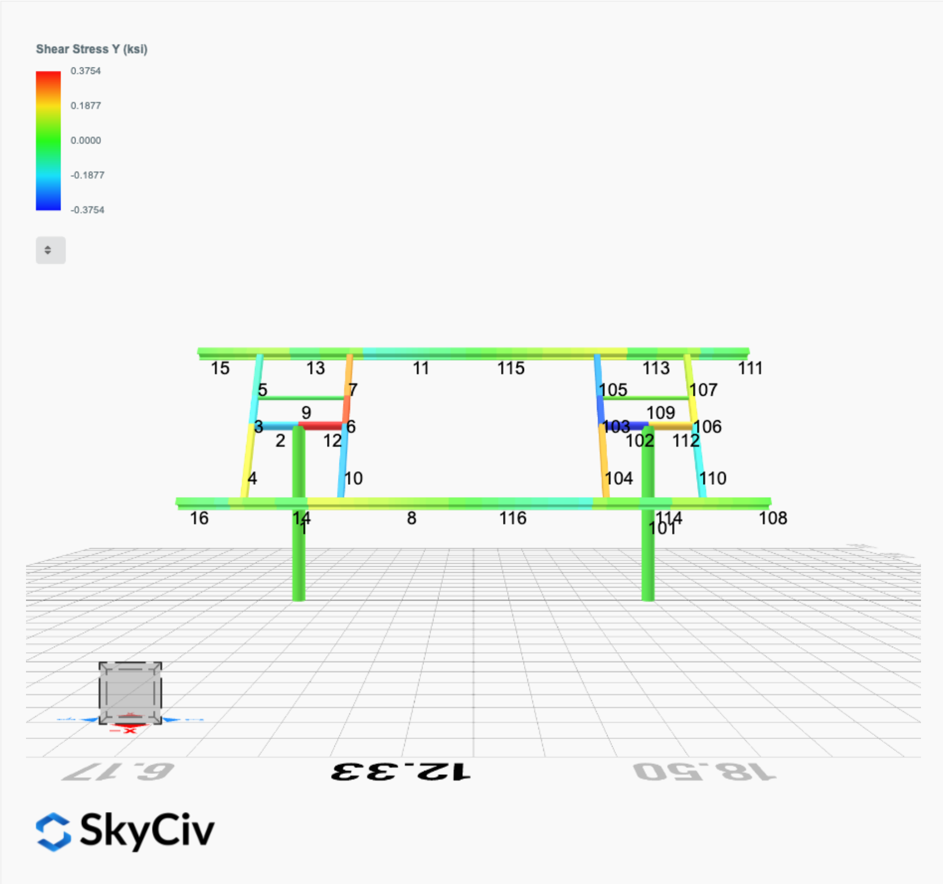
SkyCiv


Shear Stress Y (ksi)
0.3754
0.1877
0.0000
-0.1877
-0.3754
15
13
11
115
113
111
5
7
105
107
9
109
3
6
103
106
2
12
102
112
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110
16
14
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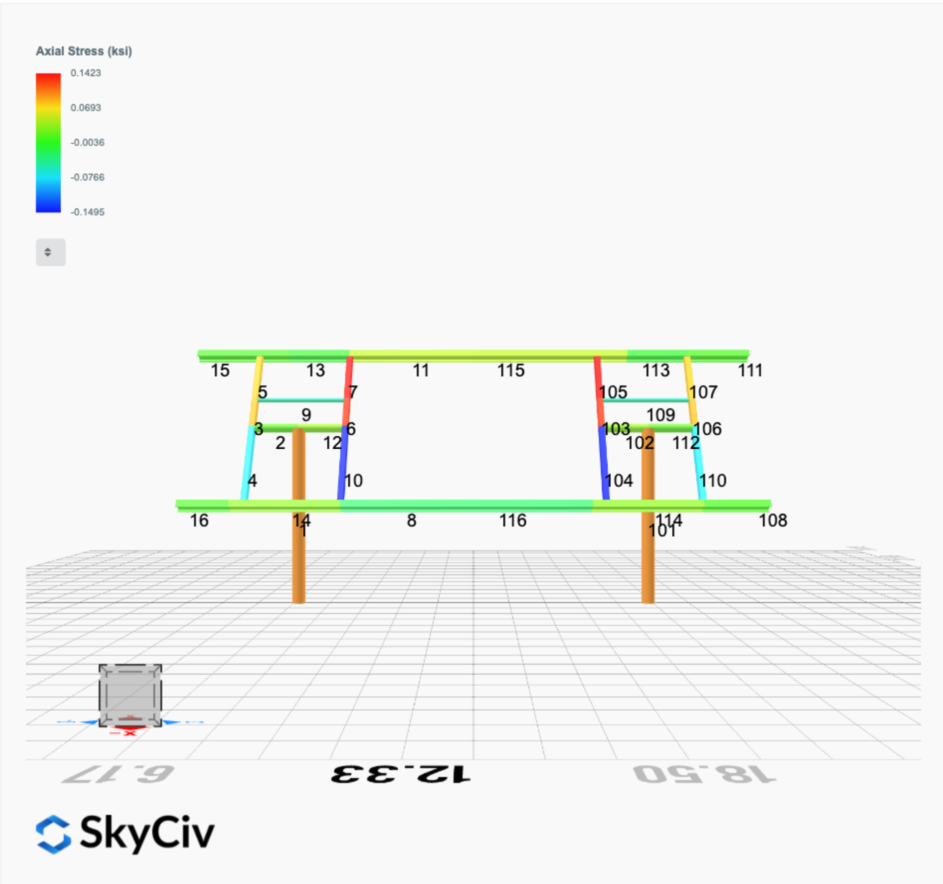
8
116
114
101
108
6.17
12.33
18.50
SkyCiv


Axial Stress (ksi)
0.1423
0.0693
-0.0036
-0.0766
-0.1495
15
13
11
115
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5
7
105
107
9
109
3
6
103
106
2
12
102
112
4
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110
16
14
1
8
116
114
101
108
6.17
12.33
18.50
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	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 2. D + L	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 3. D + (S or Lr or R)	0.0000	2.3335	0.0924	0.1750	-0.0773	0.0130
ULS: 3. D + (S or Lr or R)	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	2.1000	0.0814	0.1542	-0.0681	0.0126
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 5b. D + 0.7E	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0000	2.1000	0.0814	0.1542	-0.0681	0.0126
ULS: 8. 0.6D + 0.7E	0.0000	0.8397	0.0291	0.0551	-0.0243	0.0069
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2934	3.8783	0.2339	0.3903	-1.0563	32.5413
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.2934	-1.0792	-0.1359	-0.2049	0.9737	-31.9045
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2200	3.9591	0.2205	0.3781	-0.8300	24.4099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	2.1000	0.0814	0.1542	-0.0681	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2200	0.2409	-0.0569	-0.0683	0.6925	-23.9244
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	2.1000	0.0814	0.1542	-0.0681	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2200	3.2586	0.1876	0.3157	-0.8023	24.4088
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2200	-0.4595	-0.0898	-0.1307	0.7201	-23.9255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0000	1.3996	0.0485	0.0919	-0.0404	0.0116
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2934	3.3185	0.2145	0.3536	-1.0401	32.5366
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0000	0.8397	0.0291	0.0551	-0.0243	0.0069
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.2934	-1.6390	-0.1554	-0.2416	0.9898	-31.9091
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0000	0.8397	0.0291	0.0551	-0.0243	0.0069

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	6.2777
Shear X	-7.1556
Shear Z	0.3892
Moment X	0.6494
Moment Y (Twist)	1.7578
Moment Z	54.5554

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	3.9591
Shear X	-4.2934
Shear Z	0.2339
Moment X	0.3903
Moment Y (Twist)	1.0563
Moment Z	32.5413

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	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 2. D + L	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 3. D + (S or Lr or R)	-0.0000	2.3335	-0.0924	-0.1750	0.0773	0.0130
ULS: 3. D + (S or Lr or R)	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	2.1000	-0.0814	-0.1542	0.0681	0.0126
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 5b. D + 0.7E	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0000	2.1000	-0.0814	-0.1542	0.0681	0.0126
ULS: 8. 0.6D + 0.7E	-0.0000	0.8397	-0.0291	-0.0551	0.0243	0.0069
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.2934	3.8783	-0.2339	-0.3903	1.0563	32.5413
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.2934	-1.0792	0.1359	0.2049	-0.9737	-31.9045
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2200	3.9591	-0.2205	-0.3781	0.8300	24.4099
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	2.1000	-0.0814	-0.1542	0.0681	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2200	0.2409	0.0569	0.0684	-0.6925	-23.9244
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	2.1000	-0.0814	-0.1542	0.0681	0.0126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.2200	3.2586	-0.1876	-0.3157	0.8023	24.4088
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.2200	-0.4595	0.0898	0.1307	-0.7201	-23.9255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0000	1.3996	-0.0485	-0.0919	0.0404	0.0116
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.2934	3.3185	-0.2145	-0.3536	1.0401	32.5366
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0000	0.8397	-0.0291	-0.0551	0.0243	0.0069
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.2934	-1.6390	0.1554	0.2416	-0.9898	-31.9091
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0000	0.8397	-0.0291	-0.0551	0.0243	0.0069

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	6.2778
Shear X	-7.1556
Shear Z	-0.3892
Moment X	-0.6494
Moment Y (Twist)	1.7578
Moment Z	54.5565

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	3.9591
Shear X	-4.2934
Shear Z	-0.2339
Moment X	-0.3903
Moment Y (Twist)	1.0563
Moment Z	32.5413

Project Details

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

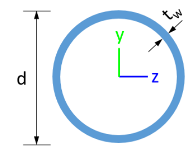
User Name: sales@mtsolar.us
Project Name: CharkowskiOffGridCabin Exp C5 CU
Unit System: imperial

Design Input Information

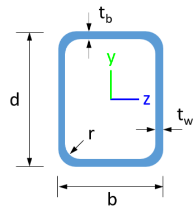
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

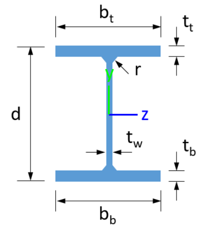
Section Dimensions



ID	Name	d (in)	t_w (in)
1	2in Pipe Sch 40	2.38	0.15
4	4in Pipe Sch 40	4.50	0.24
8	6in Pipe Sch 80	6.63	0.43



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31

8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60
15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b (ft)	C_b	LST	LSC	LD
1	8	15.76	15.76	7.50	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.17,1.18,1.17,1.19,1.17,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.17,1.19,1.17,1.19	300	200	1
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
8	18	1.33	1.33	2.05	1.20,1.20,1.20,1.20,1.20,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.18,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.67,1.68,1.67,1.69,1.67,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
11	18	1.33	1.33	2.05	1.20,1.20,1.20,1.20,1.20,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.18,1.20,1.19,1.20,1.19,1.20,1.19,1.20,1.19,1.20	300	200	1
12	4	2.00	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.43,1.44,1.43,1.45,1.43,1.43,1.62,1.44,1.66,1.44,1.62,1.43,1.66,1.43,1.59,1.45,1.74,1.45,1.61,1.43,1.68,1.43,1.63,1.43,1.65,1.43	300	200	1
14	18	4.88	4.00	7.50	1.40,1.41,1.40,1.42,1.41,1.40,1.60,1.41,1.63,1.41,1.60,1.40,1.63,1.40,1.57,1.42,1.71,1.42,1.59,1.40,1.65,1.40,1.60,1.40,1.62,1.40	300	200	1
15	18	2.10	2.10	1.00	2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
16	18	2.10	2.10	1.00	2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
101	8	15.76	15.76	7.50	-	300	200	1
102	4	2.00	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
104	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.67,1.68,1.67,1.69,1.67,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
105	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
106	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.17,1.18,1.17,1.18,1.17,1.19,1.17,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.17,1.19,1.17,1.19	300	200	1
107	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1

						0	0	
108	18	2.10	2.10	1.00	2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
109	1	2.60	2.60	4.00	-	300	200	1
110	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.67,1.69	300	200	1
111	18	2.10	2.10	1.00	2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.32	300	200	1
112	4	1.30	1.30	2.00	-	300	200	1
113	18	4.88	4.00	7.50	1.43,1.44,1.43,1.45,1.43,1.43,1.62,1.44,1.66,1.44,1.62,1.43,1.66,1.43,1.59,1.45,1.74,1.45,1.61,1.43,1.68,1.43,1.63,1.43,1.65,1.43	300	200	1
114	18	4.88	4.00	7.50	1.40,1.41,1.40,1.42,1.41,1.40,1.60,1.41,1.63,1.41,1.60,1.40,1.63,1.40,1.57,1.42,1.71,1.42,1.59,1.40,1.65,1.40,1.60,1.40,1.62,1.40	300	200	1
115	18	3.54	3.54	5.45	1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.06,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07	300	200	1
116	18	3.54	3.54	5.45	1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.06,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07,1.07	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	378.22	219.79	62.23	62.23	113.47	113.47
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	140.22	16.17	16.17	42.85	42.85
13	120.60	98.23	23.36	6.45	30.09	45.74
14	120.60	98.23	23.36	6.45	30.09	45.74
15	120.60	113.97	23.36	6.45	30.09	45.74
16	120.60	113.97	23.36	6.45	30.09	45.74
101	378.22	219.79	62.23	62.23	113.47	113.47
102	142.83	140.22	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	113.97	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	113.97	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	23.36	6.45	30.09	45.74
114	120.60	98.23	23.36	6.45	30.09	45.74

115	120.60	102.67	21.57	6.45	30.09	45.74
116	120.60	102.67	21.57	6.45	30.09	45.74

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	0.029	0.877	0.036	0.063	0.003	0.901	#13	0.431	Not Required	Pass
2	0.001	0.233	0.344	0.058	0.076	0.578	#13	0.052	Not Required	Pass
3	0.007	0.700	0.077	0.069	0.013	0.729	#13	0.044	Not Required	Pass
4	0.006	0.697	0.097	0.070	0.013	0.796	#13	0.078	Not Required	Pass
5	0.006	0.436	0.034	0.070	0.004	0.452	#13	0.073	Not Required	Pass
6	0.010	0.870	0.164	0.089	0.030	0.991	#13	0.044	Not Required	Warn
7	0.010	0.540	0.144	0.087	0.021	0.567	#13	0.073	Not Required	Pass
8	0.003	0.190	0.064	0.046	0.008	0.231	#13	0.088	Not Required	Pass
9	0.003	0.052	0.091	0.003	0.003	0.126	#13	0.132	Not Required	Pass
10	0.011	0.867	0.149	0.087	0.021	0.886	#13	0.078	Not Required	Pass
11	0.003	0.188	0.065	0.046	0.008	0.225	#13	0.088	Not Required	Pass
12	0.002	0.373	0.453	0.082	0.093	0.827	#13	0.079	Not Required	Pass
13	0.003	0.099	0.144	0.068	0.012	0.164	#21	0.265	Not Required	Pass
14	0.003	0.101	0.141	0.068	0.012	0.159	#21	0.177	Not Required	Pass
15	0.000	0.008	0.008	0.012	0.002	0.013	#13	Not Required	Not Required	Pass
16	0.000	0.008	0.008	0.012	0.002	0.013	#13	Not Required	Not Required	Pass
101	0.029	0.877	0.036	0.063	0.003	0.901	#13	0.431	Not Required	Pass
102	0.002	0.373	0.453	0.082	0.093	0.827	#13	0.079	Not Required	Pass
103	0.010	0.870	0.164	0.089	0.030	0.991	#13	0.044	Not Required	Warn
104	0.011	0.867	0.149	0.087	0.021	0.886	#13	0.078	Not Required	Pass
105	0.010	0.540	0.144	0.087	0.021	0.567	#13	0.073	Not Required	Pass
106	0.007	0.700	0.077	0.069	0.013	0.729	#13	0.044	Not Required	Pass
107	0.006	0.436	0.034	0.070	0.004	0.452	#13	0.073	Not Required	Pass
108	0.000	0.008	0.008	0.012	0.002	0.013	#13	Not Required	Not Required	Pass
109	0.003	0.052	0.091	0.003	0.003	0.126	#13	0.132	Not Required	Pass
110	0.006	0.697	0.097	0.070	0.013	0.796	#13	0.078	Not Required	Pass
111	0.000	0.008	0.008	0.012	0.002	0.013	#13	Not Required	Not Required	Pass
112	0.001	0.233	0.344	0.058	0.076	0.578	#13	0.052	Not Required	Pass
113	0.003	0.099	0.144	0.068	0.012	0.164	#21	0.177	Not Required	Pass
114	0.003	0.101	0.141	0.068	0.012	0.159	#21	0.265	Not Required	Pass
115	0.003	0.225	0.086	0.046	0.008	0.276	#13	0.235	Not Required	Pass
116	0.003	0.227	0.085	0.046	0.008	0.282	#13	0.235	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

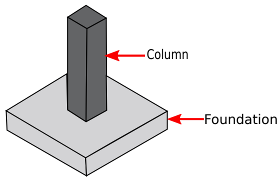
Design Information :

Isolated Foundation
Design code : ACI-318-2014 (American Concrete Institute)
Unit System : Imperial

Project Details :

Project Name : Sunlux
Client : Sunlux
Designer : Travis
Company : MT Solar
Notes :

GEOMETRY:



Foundation:
$l_z = 81\ \mathrm{in}$ - Depth,
$l_x = 81\ \mathrm{in}$ - Width,
$t = 30\ \mathrm{in}$ - Thickness,
Column:
$c_x = 18\ \mathrm{in}$ - Width,
$c_z = 18\ \mathrm{in}$ - Depth,
$h_{column} = 1\ \mathrm{in}$ - Height,
$z_{offset} = 0\ \mathrm{in}$ - Offset Along Z-axis,
$x_{offset} = 0\ \mathrm{in}$ - Offset Along X-axis,

MATERIAL PROPERTIES:

Concrete:
$f'_c = 2500\ \mathrm{psi}$ - Specified compressive strength of concrete,
Steel:
$f_y = 40000\ \mathrm{psi}$ - Specified yield strength for nonprestressed reinforcement,

Densities:
$\gamma_{soil} = 120\ \mathrm{lbf/ft^3}$ - Soil unit weight,
$\gamma_{conc} = 150\ \mathrm{lbf/ft^3}$ - Concrete unit weight,
$\gamma_{water} = 62.4\ \mathrm{lbf/ft^3}$ - Water unit weight,

SOIL PARAMETERS:

$h_{soil} = 0\ \mathrm{in}$ - Soil thickness,
$\theta_{friction} = 33$ - Angle of internal friction,
$\mu_{friction} = 0.45$ - Coefficient of friction,
$q = 0\ \mathrm{ksf}$ - Surcharge area load,
$\sigma_{bearing} = 2\ \mathrm{ksf}$ - Allowable Soil Bearing capacity (gross),

REINFORCEMENTS:

Footing:
Bottom reinforcement (x-direction): 0.375in∅; at 12in spacing
Bottom reinforcement (z-direction): 0.375in∅; at 12in spacing
Column:
Main reinforcement: 3pcs -0.375in∅;
Secondary reinforcement: 0.375in∅; at 12in spacing

MISCELLANEOUS:

REFERENCES	CALCULATIONS
Table 20.6.1.3.1	$cc_{top} = 3\ \mathrm{in}$ - Concrete cover at top,
Table 20.6.1.3.1	$cc_{bot} = 3\ \mathrm{in}$ - Concrete cover at bottom,
	$SW_{factor} = 1$ - Selfweight Factor,

GEOMETRIC VALUES:

A_{ftg} - Area of a square

$$A_{ftg} = l_z\, l_x$$

$$A_{ftg} = (81\ \mathrm{in}) \times (81\ \mathrm{in})$$

$$A_{ftg} = 6561\ \mathrm{in}^2$$

A_{col} - Area of a square

$$A_{col} = c_z\, c_x$$

$$A_{col} = (18\ \mathrm{in}) \times (18\ \mathrm{in})$$

$$A_{col} = 324 \text{ in}^2$$

V_{ftg} - Volume of foundation,

$$V_{ftg} = A_{ftg}\, t$$

$$V_{ftg} = \left(6561 \text{ in}^2\right) \times (30 \text{ in})$$

$$V_{ftg} = 113.91 \text{ ft}^3$$

V_{col} - Volume of column,

$$V_{col} = A_{col}\, h_{column}$$

$$V_{col} = \left(324 \text{ in}^2\right) \times (1 \text{ in})$$

$$V_{col} = 0.1875 \text{ ft}^3$$

SELF WEIGHTS

SW_{col} - Self weight of column,

$$SW_{col} = V_{col}\, \gamma_{conc}\, SW_{factor}$$

$$SW_{col} = \left(0.1875 \text{ ft}^3\right) \times \left(150 \text{ lbf/ft}^3\right) \times (1)$$

$$SW_{col} = 0.028125 \text{ kip}$$

SW_{ftg} - Self weight of foundation,

$$SW_{ftg} = V_{ftg}\, \gamma_{conc}\, SW_{factor}$$

$$SW_{ftg} = \left(113.91 \text{ ft}^3\right) \times \left(150 \text{ lbf/ft}^3\right) \times (1)$$

$$SW_{ftg} = 17.086 \text{ kip}$$

SW_{soil} - Self Weight of Soil,

$$SW_{soil} = \left[\left(A_{ftg}\, h_{soil}\right) - \left(A_{col}\, h_{soil}\right)\right] \gamma_{soil}\, SW_{factor}$$

$$SW_{soil} = \left[\left(\left(6561 \text{ in}^2\right) \times (0 \text{ in})\right) - \left(\left(324 \text{ in}^2\right) \times (0 \text{ in})\right)\right] \times \left(120 \text{ lbf/ft}^3\right) \times (1)$$

$$SW_{soil} = 0 \text{ kip}$$

$SW_{buoyancy} = 0 \text{ kip}$ - Total Buoyant Weight,
SW_{total} - Total weight,
Excluding column weights.

$$SW_{total} = SW_{ftg} + SW_{soil} - SW_{buoyancy}$$

$$SW_{total} = (17.086 \text{ kip}) + (0 \text{ kip}) - (0 \text{ kip})$$

$$SW_{total} = 17.086 \text{ kip}$$

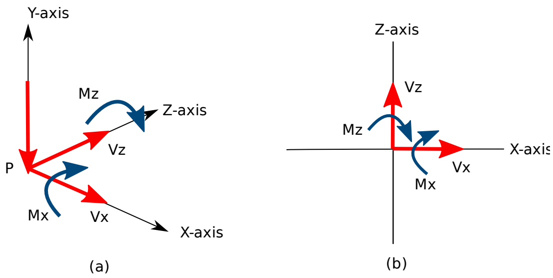
Q - Surcharge load,

$$Q = q\, A_{ftg}$$

$$Q = (0 \text{ ksf}) \times \left(6561 \text{ in}^2\right)$$

$$Q = 0 \text{ kip}$$

LOADS



Local Sign Convention

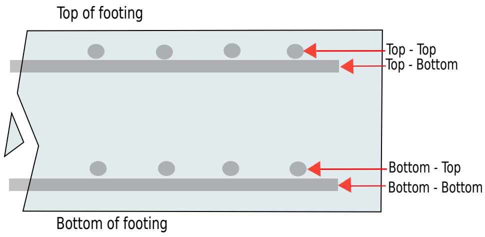
Positive direction (+) in (a) X-Y-Z axis (b) X-Z axis

Tabulation of input loads:

Load Component	Service loads	Ultimate loads
P (kip)	3.959	6.278
V_x (kip)	-4.293	-7.156

V_z (kip)	0.234	0.389
M_x (kipft)	0.390	0.649
M_z (kipft)	32.541	54.555

EFFECTIVE DEPTH



Designation of rebar positions

d_{bb} - Foundation effective depth at Bottom - Bottom

$$d_{bb} = t - cc_{bot} - \frac{\phi_{bottom,x}}{2.00}$$

$$d_{bb} = (30\text{ in}) - (3\text{ in}) - \frac{(0.375\text{ in})}{2.00}$$

$$d_{bb} = 26.813\text{ in}$$

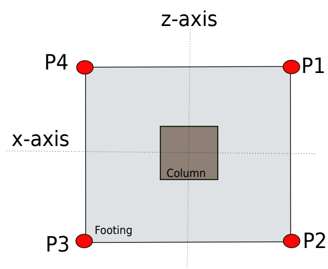
d_{bt} - Foundation effective depth at Bottom - Top

$$d_{bt} = t - cc_{bot} - \phi_{bottom,x} - \frac{\phi_{bottom,z}}{2.00}$$

$$d_{bt} = (30\text{ in}) - (3\text{ in}) - (0.375\text{ in}) - \frac{(0.375\text{ in})}{2.00}$$

$$d_{bt} = 26.438\text{ in}$$

SOIL PRESSURE CHECK



Soil Pressure (sls):

Parameters

$SW_{total,sls}$ - Factored total weight,

$$SW_{total,sls} = [DL_{sls}\ (SW_{total} + SW_{col})] + (LL_{sls}\ Q)$$

$$SW_{total,sls} = [(1) \times ((17.086\text{ kip}) + (0.028125\text{ kip}))] + ((1) \times (0\text{ kip}))$$

$$SW_{total,sls} = 17.114\text{ kip}$$

P_{sls} - Total axial load,

$$P_{sls} = P + SW_{total,sls}$$

$$P_{sls} = (3.9591\text{ kip}) + (17.114\text{ kip})$$

$$P_{sls} = 21.073\text{ kip}$$

$p_{column,sls}$ - Factored column load,

$$p_{column,sls} = P_{sls} - SW_{total,sls,net}$$

$$p_{column,sls} = (21.073\text{ kip}) - (17.086\text{ kip})$$

$$p_{column,sls} = 3.9872\text{ kip}$$

M - Total moment in the x-direction,

$$M = M_{x,sls,1} + [V_{z,sls,1}\ (h_{column} + t)] + (p_{column,sls}\ z_{offset})$$

$$M = (0.39035\text{ kipft}) + [(0.23395\text{ kip}) \times ((1\text{ in}) + (30\text{ in}))] + ((3.9872\text{ kip}) \times (0\text{ in}))$$

$$M = 0.99471\text{ kipft}$$

M_z - Total moment in the z-direction,

M - Total moment in the z-direction,

$$M = M_{z,sls,1} + [V_{x,sls,1}\ (h_{column} + t)] + (p_{column,sls}\ x_{offset})$$

$$M = (32.541\ \text{kipft}) + [(-4.2934\ \text{kip}) \times ((1\ \text{in}) + (30\ \text{in}))] + ((3.9872\ \text{kip}) \times (0\ \text{in}))$$

$$M = 21.45\ \text{kipft}$$

e_x - Eccentricity in the x-direction,

$$e_x = \frac{M}{P_{sls}}$$

$$e_x = \frac{(21.45\ \text{kipft})}{(21.073\ \text{kip})}$$

$$e_x = 1.0179\ \text{ft}$$

e_z - Eccentricity in the z-direction,

$$e_z = \frac{M}{P_{sls}}$$

$$e_z = \frac{(0.99471\ \text{kipft})}{(21.073\ \text{kip})}$$

$$e_z = 0.047203\ \text{ft}$$

The eccentricity (1.018, 0.047) falls under: **zone C**

S_x - Section modulus,

$$S_x = \frac{l_x\ l_z^2}{6}$$

$$S_x = \frac{(81\ \text{in}) \times (81\ \text{in})^2}{6}$$

$$S_x = 51.258\ \text{ft}^3$$

S_z - Section modulus,

$$S_z = \frac{l_z\ l_x^2}{6}$$

$$S_z = \frac{(81\ \text{in}) \times (81\ \text{in})^2}{6}$$

$$S_z = 51.258\ \text{ft}^3$$

P_1 - Soil pressure at

$$P_1 = \frac{P_{sls}}{A_{ftg}} + \frac{M}{S_z} + \frac{M}{S_x}$$

$$P_1 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} + \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} + \frac{(0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_1 = 0.90039\ \text{ksf}$$

P_2 - Soil pressure at

$$P_2 = \frac{P_{sls}}{A_{ftg}} + \frac{M}{S_z} - \frac{M}{S_x}$$

$$P_2 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} + \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} - \frac{(0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_2 = 0.86158\ \text{ksf}$$

P_3 - Soil pressure at

$$P_3 = \frac{P_{sls}}{A_{ftg}} - \frac{M}{S_z} - \frac{M}{S_x}$$

$$P_3 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} - \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} - \frac{(0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_3 = 0.024631\ \text{ksf}$$

P_4 - Soil pressure at

$$P_4 = \frac{P_{sls}}{A_{ftg}} - \frac{M}{S_z} + \frac{M}{S_x}$$

$$P_4 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} - \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} + \frac{(0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_4 = 0.063444\ \text{ksf}$$

q_{sls} - Maximum unfactored soil pressure,

$$q_{sls} = MAX\left[P_1, P_2, P_3, P_4\right]$$

$$q_{sls} = MAX\left[(0.90039 \text{ ksf}), (0.86158 \text{ ksf}), (0.024631 \text{ ksf}), (0.063444 \text{ ksf})\right]$$

$$q_{sls} = 0.90039 \text{ kip/ft}^2$$

Soil Pressure (uls):

Parameters

$SW_{total,uls}$ - Factored total weight,

$$SW_{total,uls} = \left[DL_{uls}\ (SW_{total} + SW_{col})\right] + (LL_{uls}\ Q)$$

$$SW_{total,uls} = \left[(1.2) \times ((17.086 \text{ kip}) + (0.028125 \text{ kip}))\right] + ((1.6) \times (0 \text{ kip}))$$

$$SW_{total,uls} = 20.537 \text{ kip}$$

P_{uls} - Total axial load,

$$P_{uls} = P + SW_{total,uls}$$

$$P_{uls} = (6.2777 \text{ kip}) + (20.537 \text{ kip})$$

$$P_{uls} = 26.815 \text{ kip}$$

$p_{column,uls}$ - Factored column load,

$$p_{column,uls} = P_{uls} - SW_{total,uls,net}$$

$$p_{column,uls} = (26.815 \text{ kip}) - (20.503 \text{ kip})$$

$$p_{column,uls} = 6.3115 \text{ kip}$$

M - Total moment in the x-direction,

$$M = M_{x,uls,1} + \left[V_{z,uls,1}\ (h_{column} + t)\right] + (p_{column,uls}\ z_{offset})$$

$$M = (0.64941 \text{ kipft}) + \left[(0.38917 \text{ kip}) \times ((1 \text{ in}) + (30 \text{ in}))\right] + ((6.3115 \text{ kip}) \times (0 \text{ in}))$$

$$M = 1.6547 \text{ kipft}$$

M - Total moment in the z-direction,

$$M = M_{z,uls,1} + \left[V_{x,uls,1}\ (h_{column} + t)\right] + (p_{column,uls}\ x_{offset})$$

$$M = (54.555 \text{ kipft}) + \left[(-7.1556 \text{ kip}) \times ((1 \text{ in}) + (30 \text{ in}))\right] + ((6.3115 \text{ kip}) \times (0 \text{ in}))$$

$$M = 36.07 \text{ kipft}$$

e_x - Eccentricity in the x-direction,

$$e_x = \frac{M}{P_{uls}}$$

$$e_x = \frac{(36.07 \text{ kipft})}{(26.815 \text{ kip})}$$

$$e_x = 1.3452 \text{ ft}$$

e_z - Eccentricity in the z-direction,

$$e_z = \frac{M}{P_{uls}}$$

$$e_z = \frac{(1.6547 \text{ kipft})}{(26.815 \text{ kip})}$$

$$e_z = 0.061711 \text{ ft}$$

The eccentricity (1.345, 0.062) falls under: **zone E**

P_1

$$P_1 = \frac{P_{uls}}{3\, l_x\, l_z} \left(2 - \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \left(1 + \frac{6\, e_z}{l_z} + \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \frac{1}{\frac{1}{2} - \frac{e_x}{l_x}}$$

$$P_1 = \frac{(26.815)}{3 \times (81) \times (81)} \times \left(2 - \sqrt{1 - \frac{12 \times (0.061711)^2}{(81)^2}}\right) \times \left(1 + \frac{6 \times (0.061711)}{(81)} + \sqrt{1 - \frac{12 \times (0.061711)^2}{(81)^2}}\right) \times \frac{1}{\frac{1}{2} - \frac{(1.3452)}{(81)}}$$

$$P_1 = 1.3408 \text{ kip/ft}^2$$

P_2

$$P_2 = \frac{P_{uls}}{3\, l_x\, l_z} \left(2 - \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \left(1 - \frac{6\, e_z}{l_z} + \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \frac{1}{\frac{1}{2} - \frac{e_x}{l_x}}$$

$$P_2 = \frac{(26.815)}{3 \times (81) \times (81)} \times \left(2 - \sqrt{1 - \frac{12 \times (0.061711)^2}{(81)^2}}\right) \times \left(1 - \frac{6 \times (0.061711)}{(81)} + \sqrt{1 - \frac{12 \times (0.061711)^2}{(81)^2}}\right) \times \frac{1}{\frac{1}{2} - \frac{(1.3452)}{(81)}}$$

$$P_2 = 1.2692\ \mathrm{kip/ft^2}$$

$P_3 = 0\ \mathrm{kip/ft^2}$ - Soil pressure at

$P_4 = 0\ \mathrm{kip/ft^2}$ - Soil pressure at

q_{uls} - Maximum factored soil pressure,

$$q_{uls} = MAX\left[P_1, P_2, P_3, P_4\right]$$

$$q_{uls} = MAX\left[\left(1.3408\ \mathrm{kip/ft^2}\right), \left(1.2692\ \mathrm{kip/ft^2}\right), \left(0\ \mathrm{kip/ft^2}\right), \left(0\ \mathrm{kip/ft^2}\right)\right]$$

$$q_{uls} = 1.3408\ \mathrm{kip/ft^2}$$

Summary:

Corner Pressure	p1	p2	p3	p4
Service	0.900	0.862	0.025	0.063
Ultimate	1.341	1.269	0.000	0.000

Check soil bearing capacity ratio:

$Ratio$

$$Ratio = \frac{q_{sls}}{\sigma_{bearing}}$$

$$Ratio = \frac{\left(0.90039\ \mathrm{kip/ft^2}\right)}{(2\ \mathrm{ksf})}$$

$$Ratio = 0.4502$$

Status: **PASS**
Ratio = **0.450**

UPLIFT CHECK

Uplift stability considers axial loads only

Governing SLS axial force = 21.073 kip

∴ Uplift does not exist!

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

SLIDING CHECK

$\phi_{friction}$ - Angle of internal friction (rad),

$$\phi_{friction} = \theta_{friction}\ \frac{\pi}{180}$$

$$\phi_{friction} = (33) \times \frac{\pi}{180}$$

$$\phi_{friction} = 0.57596$$

k_p - Passive Coefficient,

$$k_p = \frac{1 + \sin\phi_{friction}}{1 - \sin\phi_{friction}}$$

$$k_p = \frac{1 + \sin(0.57596)}{1 - \sin(0.57596)}$$

$$k_p = 3.3921$$

P_{mid} - Pressure at mid depth,

$$P_{mid} = k_p\ \gamma_{soil}\ (h_{soil} + \frac{t}{2})$$

$$P_{mid} = (3.3921) \times \left(120\ \mathrm{lbf/ft^3}\right) \times ((0\ \mathrm{in}) + \frac{(30\ \mathrm{in})}{2})$$

$$P_{mid} = 0.50882\ \mathrm{ksf}$$

Sliding check along the x-direction:

$F_{p,x}$ - Passive Force,

$$F_{p,x} = P_{mid}\ t\ l_z$$

$$F_{p,x} = (0.50882\ \mathrm{ksf}) \times (30\ \mathrm{in}) \times (81\ \mathrm{in})$$

$$F_{p,x} = 8.5863\ \mathrm{kip}$$

F_f - Friction Force,

$$F_f = \mu_{friction}\ P_{sls}$$

$$F_f = (0.45) \times (21.073\ \mathrm{kip})$$

$$F_f = 9.4829 \text{ kip}$$

FOS_{actual} - Actual Factor of Safety for Sliding,

$$FOS_{actual} = \frac{F_{p,x} + F_f}{V_{x,sls,1}}$$

$$FOS_{actual} = \frac{(8.5863 \text{ kip}) + (9.4829 \text{ kip})}{(-4.2934 \text{ kip})}$$

$$FOS_{actual} = -4.2086$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(1.5)}{(-4.2086)}$$

$$Ratio = 0.35641$$

Status: **PASS**
Ratio = **0.356**

Sliding check along the z-direction:

$F_{p,z}$ - Passive Force,

$$F_{p,z} = P_{mid}\, t\, l_x$$

$$F_{p,z} = (0.50882 \text{ ksf}) \times (30 \text{ in}) \times (81 \text{ in})$$

$$F_{p,z} = 8.5863 \text{ kip}$$

F_f - Friction Force,

$$F_f = \mu_{friction}\, P_{sls}$$

$$F_f = (0.45) \times (21.073 \text{ kip})$$

$$F_f = 9.4829 \text{ kip}$$

FOS_{actual} - Actual Factor of Safety for Sliding,

$$FOS_{actual} = \frac{F_{p,z} + F_f}{V_{z,sls,1}}$$

$$FOS_{actual} = \frac{(8.5863 \text{ kip}) + (9.4829 \text{ kip})}{(0.23395 \text{ kip})}$$

$$FOS_{actual} = 77.236$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(1.5)}{(77.236)}$$

$$Ratio = 0.019421$$

Status: **PASS**
Ratio = **0.019**

OVERTURNING CHECK

Overturning along the x-direction:

FOS_{actual} - Actual Factor of Safety for Overturning,

$$FOS_{actual} = \frac{\left(\frac{l_x}{2}\right)}{e_x}$$

$$FOS_{actual} = \frac{\left(\frac{(81 \text{ in})}{2}\right)}{(1.0179 \text{ ft})}$$

$$FOS_{actual} = 3.3157$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(2)}{(3.3157)}$$

$$Ratio = 0.60319$$

Status: **PASS**
Ratio = **0.603**

Overturning along the z-direction:

FOS_{actual} - Actual Factor of Safety for Overturning,

$$FOS_{actual} = \frac{\left(\frac{l_z}{2}\right)}{e_z}$$

$$FOS_{actual} = \frac{\left(\frac{(81\text{ in})}{2}\right)}{(0.047203\text{ ft})}$$

$$FOS_{actual} = 71.5$$

Ratio

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(2)}{(71.5)}$$

$$Ratio = 0.027972$$

Status: **PASS**
Ratio = **0.028**

ONE WAY SHEAR CHECK

Table 21.2.2 $\theta_{shear} = 0.75$ - Shear reduction factor,

d - Critical depth,

$$d = \frac{d_{bb} + d_{bt}}{2}$$

$$d = \frac{(26.813\text{ in}) + (26.438\text{ in})}{2}$$

$$d = 26.625\text{ in}$$

Critical section along the x-direction:

20.6.1.3.1 $V_{demand,x}$ - Shear demand,

$$V_{demand,x} = q_{uls}\left(\frac{l_z}{2} + |z_{offset}| - \frac{c_z}{2} - d\right) l_x$$

$$V_{demand,x} = \left(1.3408\text{ kip/ft}^2\right) \times \left(\frac{(81\text{ in})}{2} + |\,(0\text{ in})\,| - \frac{(18\text{ in})}{2} - (26.625\text{ in})\right) \times (81\text{ in})$$

$$V_{demand,x} = 3.6768\text{ kip}$$

$V_{capacity,x}$ - Shear Capacity

$$V_{capacity,x} = \theta_{shear}\ 2\ \lambda\ \sqrt{f_c'}\ l_z\ d$$

$$V_{capacity,x} = (0.75) \times 2 \times (1) \times \sqrt{(2500\text{ psi})} \times (81\text{ in}) \times (26.625\text{ in})$$

$$V_{capacity,x} = 161750\text{ lbf}$$

Critical section along the z-direction:

20.6.1.3.1 $V_{demand,z}$ - Shear demand,

$$V_{demand,z} = q_{uls}\left(\frac{l_x}{2} + |x_{offset}| - \frac{c_x}{2} - d\right) l_z$$

$$V_{demand,z} = \left(1.3408\text{ kip/ft}^2\right) \times \left(\frac{(81\text{ in})}{2} + |\,(0\text{ in})\,| - \frac{(18\text{ in})}{2} - (26.625\text{ in})\right) \times (81\text{ in})$$

$$V_{demand,z} = 3.6768\text{ kip}$$

$V_{capacity,z}$ - Shear Capacity

$$V_{capacity,z} = \theta_{shear}\ 2\ \lambda\ \sqrt{f_c'}\ l_x\ d$$

$$V_{capacity,z} = (0.75) \times 2 \times (1) \times \sqrt{(2500\text{ psi})} \times (81\text{ in}) \times (26.625\text{ in})$$

$$V_{capacity,z} = 161750\text{ lbf}$$

Governing shear capacity and demand:

V_{demand} - Governing shear demand,

$$V_{demand} = MAX\left[V_{demand,x}, V_{demand,z}\right]$$

$$V_{demand} = MAX\left[(3.6768\text{ kip}), (3.6768\text{ kip})\right]$$

$$V_{demand} = 3.6768\text{ kip}$$

$V_{capacity}$ - Governing shear capacity,

$$V_{capacity} = MIN\left[V_{capacity,x}, V_{capacity,z}\right]$$

$$V_{capacity} = MIN\left[(161750\ \mathrm{lbf}), (161750\ \mathrm{lbf})\right]$$

$$V_{capacity} = 161.75\ \mathrm{kip}$$

Check capacity ratio:
$Ratio$

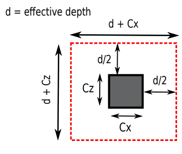
$$Ratio = \frac{V_{demand}}{V_{capacity}}$$

$$Ratio = \frac{(3.6768\ \mathrm{kip})}{(161.75\ \mathrm{kip})}$$

$$Ratio = 0.022732$$

Status: **PASS**
Ratio = **0.023**

TWO WAY SHEAR CHECK



PLAN

Parameters:
22.6.5.3 $\alpha_s = 40$ - Column position factor (interior)
$\lambda = 1$ - Concrete weight type factor,
22.6.4.1 b_o - Critical shear perimeter,

$$b_o = \left[(d + c_x) + (d + c_z)\right]\ 2$$

$$b_o = \left[((26.625\ \mathrm{in}) + (18\ \mathrm{in})) + ((26.625\ \mathrm{in}) + (18\ \mathrm{in}))\right] \times 2$$

$$b_o = 178.5\ \mathrm{in}$$

$A_{punching}$ - Punching shear area,

$$A_{punching} = (d + c_x)\ (d + c_z)$$

$$A_{punching} = ((26.625\ \mathrm{in}) + (18\ \mathrm{in})) \times ((26.625\ \mathrm{in}) + (18\ \mathrm{in}))$$

$$A_{punching} = 1991.4\ \mathrm{in}^2$$

$V_{tw,demand}$ - Punching shear demand

$$V_{tw,demand} = (A_{ftg} - A_{punching})\ q_{uls}$$

$$V_{tw,demand} = \left((6561\ \mathrm{in}^2) - (1991.4\ \mathrm{in}^2)\right) \times \left(1.3408\ \mathrm{kip/ft}^2\right)$$

$$V_{tw,demand} = 42.55\ \mathrm{kip}$$

22.6.5.2 (a) $V_{capacity,a}$ - Two-way shear capacity (a)

$$V_{capacity,a} = 4\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,a} = 4 \times (1) \times \sqrt{(2500\ \mathrm{psi})} \times (178.5\ \mathrm{in}) \times (26.625\ \mathrm{in})$$

$$V_{capacity,a} = 950510\ \mathrm{lbf}$$

β - Ratio of the long side to short side of column,

$$\beta = \frac{MAX(c_x, c_z)}{MIN(c_x, c_z)}$$

$$\beta = \frac{MAX((18\ \mathrm{in}), (18\ \mathrm{in}))}{MIN((18\ \mathrm{in}), (18\ \mathrm{in}))}$$

$$\beta = 1$$

22.6.5.2 (b) $V_{capacity,b}$ - Two-way shear capacity {b}

$$V_{capacity,b} = \left(2 + \frac{4}{\beta}\right)\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,b} = \left(2 + \frac{4}{(1)}\right) \times (1) \times \sqrt{(2500\ \mathrm{psi})} \times (178.5\ \mathrm{in}) \times (26.625\ \mathrm{in})$$

$$V_{capacity,b} = 1.4258{\times}10^{6}\ \mathrm{lbf}$$

22.6.5.2 (c) $V_{capacity,c}$ - Two-way shear capacity (c)

$$V_{capacity,c} = \left(2 + \frac{\alpha_s\ d}{b_o}\right)\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,c} = \left(2 + \frac{(40) \times (26.625\ \text{in})}{(178.5\ \text{in})}\right) \times (1) \times \sqrt{(2500\ \text{psi})} \times (178.5\ \text{in}) \times (26.625\ \text{in})$$

$$V_{capacity,c} = 1.893{\times}10^{6}\ \text{lbf}$$

$V_{tw,capacity}$ - Governing shear capacity,

$$V_{tw,capacity} = MIN\left[V_{capacity,a}, V_{capacity,b}, V_{capacity,c}\right]$$

$$V_{tw,capacity} = MIN\left[(950510\ \text{lbf}), (1.4258{\times}10^{6}\ \text{lbf}), (1.893{\times}10^{6}\ \text{lbf})\right]$$

$$V_{tw,capacity} = 950.51\ \text{kip}$$

Check capacity ratio:

$Ratio$

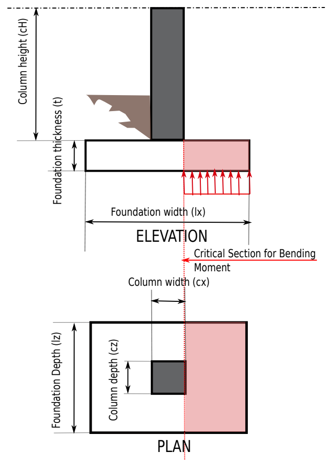
$$Ratio = \frac{V_{tw,demand}}{V_{tw,capacity}}$$

$$Ratio = \frac{(42.55\ \text{kip})}{(950.51\ \text{kip})}$$

$$Ratio = 0.044765$$

Status: **PASS**
Ratio = **0.045**

FLEXURE CHECK



Flexural check along the x-direction:

$M_{demand,x}$ - Moment demand

$$M_{demand,x} = q_{uls}\left(\frac{l_x - c_x}{2} + |x_{offset}|\right) l_z \frac{\left(\frac{l_x - c_x}{2} + |x_{offset}|\right)}{2}$$

$$M_{demand,x} = \left(1.3408\ \text{kip/ft}^2\right) \times \left(\frac{(81\ \text{in}) - (18\ \text{in})}{2} + |(0\ \text{in})|\right) \times (81\ \text{in}) \times \frac{\left(\frac{(81\ \text{in}) - (18\ \text{in})}{2} + |(0\ \text{in})|\right)}{2}$$

$$M_{demand,x} = 31.182\ \text{kipft}$$

n - No. of reinforcement

$$n = \left[\frac{l_z - (2\,cc_{bot}) - \phi_{bottom,x}}{s_{bottom,x}}\right] + 1$$

$$n = \left[\frac{(81\ \text{in}) - (2 \times (3\ \text{in})) - (0.375\ \text{in})}{(12\ \text{in})}\right] + 1$$

$$n = 8$$

A_g - Area gross of the foundation

$$A_g = t\, l_z$$

$$A_g = (30\ \text{in}) \times (81\ \text{in})$$

$$A_g = 2430\ \text{in}^2$$

Table 8.6.1.1 $A_{s,min}$ - Minimum area of reinforcement

$$A_{s,min} = 0.0020\, A_g$$

$$A_{s,min} = 0.0020 \times \left(2430\ \text{in}^2\right)$$

$$A_{s,min} = 4.86\ \text{in}^2$$

A_{rebar} - Reinforcement area

$$A_{rebar} = \frac{\pi}{4}\left(\phi_{bottom,x}\right)^2$$

$$A_{rebar} = \frac{\pi}{4} \times \left((0.375\ \text{in})\right)^2$$

$$A_{rebar} = 0.11045\ \text{in}^2$$

$A_{s,prov}$ - Total reinforcement area

$$A_{s,prov} = n\,\frac{\pi\,\phi_{bottom,x}^2}{4}$$

$$A_{s,prov} = (8) \times \frac{\pi \times (0.375\ \text{in})^2}{4}$$

$$A_{s,prov} = 0.88357\ \text{in}^2$$

Check reinforcement ratio:
$Ratio$

$$Ratio = \frac{A_{s,min}}{A_{s,prov}}$$

$$Ratio = \frac{\left(4.86\ \text{in}^2\right)}{\left(0.88357\ \text{in}^2\right)}$$

$$Ratio = 5.5004$$

Status: **FAIL**
Ratio = **5.500**

Since 2500 psi $\leq f'_c \leq$ 4000 psi:
Table 22.2.2.4.3 $\beta_1 = 0.85$ - Equivalent rectangular concrete stress distribution
a - Depth of equivalent rectangular stress block,

$$a = \frac{A_{s,prov}\,f_y}{0.85\,f'_c\,l_z}$$

$$a = \frac{\left(0.88357\ \text{in}^2\right) \times (40000\ \text{psi})}{0.85 \times (2500\ \text{psi}) \times (81\ \text{in})}$$

$$a = 0.20533\ \text{in}$$

22.2.2.4.2 c - Distance from the extreme compression fiber to neutral axis,

$$c = \frac{a}{\beta_1}$$

$$c = \frac{(0.20533\ \text{in})}{(0.85)}$$

$$c = 0.24157\ \text{in}$$

21.2.2 $\varepsilon_c = 0.003$ - Compressive strain,
21.2.2 ε_t - Tensile strain in the reinforcement,

$$\varepsilon_t = \frac{\varepsilon_c}{c}\,(d - c)$$

$$\varepsilon_t = \frac{(0.003)}{(0.24157\ \text{in})} \times \left((26.625\ \text{in}) - (0.24157\ \text{in})\right)$$

$$\varepsilon_t = 0.32765$$

Since $\varepsilon_t \geq 0.005$ (Tension-controlled):
Table 21.2.2 $\phi = 0.9$ - Strength reduction factor:,
$M_{capacity,x}$ - Moment Capacity

$$M_{capacity,x} = \phi\,A_{s,prov}\,f_y\,\left(d - \frac{a}{2}\right)$$

$$M_{capacity,x} = (0.9) \times \left(0.88357\ \text{in}^2\right) \times (40000\ \text{psi}) \times \left((26.625\ \text{in}) - \frac{(0.20533\ \text{in})}{2}\right)$$

$$M_{capacity,x} = 70.303\ \text{kipft}$$

Flexural check along the z-direction:
$M_{demand,z}$ - Moment demand

$$M_{demand,z} = q_{uls}\,\left(\frac{l_z - c_z}{2} + |z_{offset}|\right)\,l_x$$

$$\left(\frac{l_z - c_z}{2} + |z_{offset}|\right)$$

$$\frac{\left(\cdots \right)}{2}$$

$$M_{demand,z} = \left(1.3408\ \mathrm{kip/ft^2}\right) \times \left(\frac{(81\ \mathrm{in}) - (18\ \mathrm{in})}{2} + |\,(0\ \mathrm{in})\,| \right) \times (81\ \mathrm{in}) \times \frac{\left(\frac{(81\ \mathrm{in}) - (18\ \mathrm{in})}{2} + |\,(0\ \mathrm{in})\,| \right)}{2}$$

$$M_{demand,z} = 31.182\ \mathrm{kipft}$$

n - No. of reinforcement

$$n = \left[\frac{l_x - (2\ cc_{bot}) - \phi_{bottom,z}}{s_{bottom,z}} \right] + 1$$

$$n = \left[\frac{(81\ \mathrm{in}) - (2 \times (3\ \mathrm{in})) - (0.375\ \mathrm{in})}{(12\ \mathrm{in})} \right] + 1$$

$$n = 8$$

A_g - Area gross of the foundation

$$A_g = t\ l_x$$

$$A_g = (30\ \mathrm{in}) \times (81\ \mathrm{in})$$

$$A_g = 2430\ \mathrm{in}^2$$

Table 8.6.1.1 $A_{s,min}$ - Minimum area of reinforcement

$$A_{s,min} = 0.0020\ A_g$$

$$A_{s,min} = 0.0020 \times \left(2430\ \mathrm{in}^2\right)$$

$$A_{s,min} = 4.86\ \mathrm{in}^2$$

A_{rebar} - Reinforcement area

$$A_{rebar} = \frac{\pi}{4}\left(\phi_{bottom,z}\right)^2$$

$$A_{rebar} = \frac{\pi}{4} \times \left((0.375\ \mathrm{in})\right)^2$$

$$A_{rebar} = 0.11045\ \mathrm{in}^2$$

$A_{s,prov}$ - Total reinforcement area

$$A_{s,prov} = n\ \frac{\pi\ \phi_{bottom,z}^2}{4}$$

$$A_{s,prov} = (8) \times \frac{\pi \times (0.375\ \mathrm{in})^2}{4}$$

$$A_{s,prov} = 0.88357\ \mathrm{in}^2$$

Check reinforcement ratio:

Ratio

$$Ratio = \frac{A_{s,min}}{A_{s,prov}}$$

$$Ratio = \frac{\left(4.86\ \mathrm{in}^2\right)}{\left(0.88357\ \mathrm{in}^2\right)}$$

$$Ratio = 5.5004$$

Status: **FAIL**
Ratio = **5.500**

Since 2500 psi $\leq f'_c \leq$ 4000 psi:

Table 22.2.2.4.3 $\beta_1 = 0.85$ - Equivalent rectangular concrete stress distribution

a - Depth of equivalent rectangular stress block,

$$a = \frac{A_{s,prov}\ f_y}{0.85\ f'_c\ l_x}$$

$$a = \frac{\left(0.88357\ \mathrm{in}^2\right) \times (40000\ \mathrm{psi})}{0.85 \times (2500\ \mathrm{psi}) \times (81\ \mathrm{in})}$$

$$a = 0.20533\ \mathrm{in}$$

22.2.2.4.2 c - Distance from the extreme compression fiber to neutral axis,

$$c = \frac{a}{\beta_1}$$

$$c = \frac{(0.20533\ \mathrm{in})}{(0.85)}$$

21.2.2 *21.2.2* *Table 21.2.2*	$$c = 0.24157 \text{ in}$$ $\varepsilon_c = 0.003$ - Compressive strain, ε_t - Tensile strain in the reinforcement, $$\varepsilon_t = \frac{\varepsilon_c}{c}(d - c)$$ $$\varepsilon_t = \frac{(0.003)}{(0.24157 \text{ in})} \times ((26.625 \text{ in}) - (0.24157 \text{ in}))$$ $$\varepsilon_t = 0.32765$$ Since $\varepsilon_t \geq 0.005$ (Tension-controlled): $\phi = 0.9$ - Strength reduction factor:, $M_{capacity,z}$ - Moment Capacity $$M_{capacity,z} = \phi\, A_{s,prov}\, f_y \left(d - \frac{a}{2}\right)$$ $$M_{capacity,z} = (0.9) \times \left(0.88357 \text{ in}^2\right) \times (40000 \text{ psi}) \times \left((26.625 \text{ in}) - \frac{(0.20533 \text{ in})}{2}\right)$$ $$M_{capacity,z} = 70.303 \text{ kipft}$$ **Governing moment capacity and demand:** M_{demand} - Governing moment demand, $$M_{demand} = MAX\left[M_{demand,x}, M_{demand,z}\right]$$ $$M_{demand} = MAX\left[(31.182 \text{ kipft}), (31.182 \text{ kipft})\right]$$ $$M_{demand} = 31.182 \text{ kipft}$$ $M_{capacity}$ - Governing moment capacity, $$M_{capacity} = MIN\left[M_{capacity,x}, M_{capacity,z}\right]$$ $$M_{capacity} = MIN\left[(70.303 \text{ kipft}), (70.303 \text{ kipft})\right]$$ $$M_{capacity} = 70.303 \text{ kipft}$$ Check capacity ratio: $Ratio$ $$Ratio = \frac{M_{demand}}{M_{capacity}}$$ $$Ratio = \frac{(31.182 \text{ kipft})}{(70.303 \text{ kipft})}$$ $$Ratio = 0.44354$$	Status: **PASS** Ratio = **0.444**
        21.2.1(d) *Table 22.8.3.2*	**LOAD TRANSFER OF COLUMN FORCES TO THE BASE** **Nominal bearing strength:** ϕB_n $$\phi B_n = MIN\left[\left(\phi \times \sqrt{\frac{A_1}{A_2}} \times 0.85 \times f'_c \times A_1\right), \left(\phi \times 2\left(\times 0.85 \times f'_c \times A_1\right)\right)\right]$$ Where: $A_1 = 324 \text{ in}^2$ - Bearing area of column, $A_2 = 6561 \text{ in}^2$ - Footing area, $\phi = 0.65$ - Bearing strength reduction factor, ϕB_n $$\phi B_n = MIN\left[\left(\phi \sqrt{\frac{A_1}{A_2}}\, 0.85\, f'_c\, A_1\right), \left(\phi\, 2\left(0.85\, f'_c\, A_1\right)\right)\right]$$ $$\phi B_n = MIN\left[\left((0.65) \times \sqrt{\frac{(324 \text{ in}^2)}{(6561 \text{ in}^2)}} \times 0.85 \times (2500 \text{ psi}) \times \left(324 \text{ in}^2\right)\right), \left((0.65) \times 2\left(\times 0.85 \times (2500 \text{ psi}) \times \left(324 \text{ in}^2\right)\right)\right)\right]$$ $$\phi B_n = 895.05 \text{ kip}$$ **Check capacity ratio:** $Ratio$	

$$Ratio = \frac{P_{axial}}{\phi B_n}$$

$$Ratio = \frac{(26.815\ \text{kip})}{(895.05\ \text{kip})}$$

$$Ratio = 0.02996$$

Status: **PASS**
Ratio = **0.030**

DEVELOPMENT LENGTH

Compression development length (column):

$\psi_r = 1$ - Confining reinforcement factor,

Compression development length shall be the greater of the following $l_{dc,1}$, $l_{dc,2}$, 8 in :

25.4.9.1 $l_{dc,1}$

$$l_{dc,1} = \phi_{column}\ \frac{f_y\ \psi_r}{50\ \lambda\ \sqrt{f'_c}}$$

$$l_{dc,1} = (0.375\ \text{in}) \times \frac{(40000\ \text{psi}) \times (1)}{50 \times (1) \times \sqrt{(2500\ \text{psi})}}$$

$$l_{dc,1} = 6\ \text{in}$$

25.4.9.1 $l_{dc,2}$

$$l_{dc,2} = 0.0003\ f_y\ \psi_r\ \phi_{column}$$

$$l_{dc,2} = 0.0003 \times (40000\ \text{psi}) \times (1) \times (0.375\ \text{in})$$

$$l_{dc,2} = 4.5\ \text{in}$$

25.4.9.1 l_{dc}

$$l_{dc} = MAX\left[l_{dc,1}, l_{dc,2}, 8\ \text{in}\right]$$

$$l_{dc} = MAX\left[(6\ \text{in}), (4.5\ \text{in}), (8\ \text{in})\right]$$

$$l_{dc} = 8\ \text{in}$$

Table 25.3.2 r - Minimum bend diameter,

$$r = 4\ \phi_{column}$$

$$r = 4 \times (0.375\ \text{in})$$

$$r = 1.5\ \text{in}$$

$h_{req'd}$ - Required depth,

$$h_{req'd} = cc_{bot} + \phi_{bottom,x} + \phi_{bottom,z} + \phi_{column} + r + l_{dc}$$

$$h_{req'd} = (3\ \text{in}) + (0.375\ \text{in}) + (0.375\ \text{in}) + (0.375\ \text{in}) + (1.5\ \text{in}) + (8\ \text{in})$$

$$h_{req'd} = 13.625\ \text{in}$$

Check footing depth provided:

$t = 30\ \text{in}$ - Thickness,

$Ratio$

$$Ratio = \frac{h_{req'd}}{t}$$

$$Ratio = \frac{(13.625\ \text{in})}{(30\ \text{in})}$$

$$Ratio = 0.45417$$

Status: **PASS**
Ratio = **0.454**

Flexural reinforcement bar development:

l_d - Bar development length,

$$l_d = \left[\frac{3 \times f_y}{40 \times \lambda \times \sqrt{f'_c}} \times \frac{\psi_t \times \psi_e \times \psi_s}{\frac{c_b + K_{tr}}{d_b}}\right] \times d_b$$

Parameters:

Table 25.4.2.4 $\psi_t = 1$ - Casting position factor,

Table 25.4.2.4 $\psi_e = 1$ - Coating factor,

Table 25.4.2.4 $\psi_s = 0.8$ - Bar size factor

25.4.2.3 (b) $K_{tr} = 0$ in - Transverse reinforcement index,

25.4.2.3 c_b

$$c_b = MIN\left[\left(cc_{bot} + \frac{\phi_{bottom,x}}{2}\right), \left(cc_{bot} + \frac{\phi_{bottom,z}}{2}\right), \frac{s_{bottom,x}}{2}, \frac{s_{bottom,z}}{2}\right]$$

	$$c_b = MIN\left[\left((3\text{ in}) + \frac{(0.375\text{ in})}{2}\right), \left((3\text{ in}) + \frac{(0.375\text{ in})}{2}\right), \frac{(12\text{ in})}{2}, \frac{(12\text{ in})}{2}\right]$$ $$c_b = 3.1875\text{ in}$$	
25.4.2.3	$\frac{c_b+K_{tr}}{d_b}$ - Confinement term $$\frac{c_b + K_{tr}}{d_b} = MIN\left[\frac{c_b + K_{tr}}{\phi_{column}}, 2.5\right]$$ $$\frac{c_b + K_{tr}}{d_b} = MIN\left[\frac{(3.1875\text{ in}) + (0\text{ in})}{(0.375\text{ in})}, 2.5\right]$$ $$\frac{c_b + K_{tr}}{d_b} = 2.5$$	
25.4.2.3	l_d - Bar development length, $$l_d = \left[\frac{3\ f_y}{40\ \lambda\ \sqrt{f'_c}}\ \frac{\psi_t\ \psi_e\ \psi_s}{\frac{c_b+K_{tr}}{d_b}}\right]\ \phi_{column}$$ $$l_d = \left[\frac{3 \times (40000\text{ psi})}{40 \times (1) \times \sqrt{(2500\text{ psi})}} \times \frac{(1) \times (1) \times (0.8)}{(2.5)}\right] \times (0.375\text{ in})$$ $$l_d = 7.2\text{ in}$$	
	Development length provided along x-direction: $l_{d,prov'd}$ - Development length provided, $$l_{d,prov'd} = \frac{l_x - c_x}{2} - cc_{bot}$$ $$l_{d,prov'd} = \frac{(81\text{ in}) - (18\text{ in})}{2} - (3\text{ in})$$ $$l_{d,prov'd} = 28.5\text{ in}$$	
	Check development length ratio: $Ratio$ $$Ratio = \frac{l_d}{l_{d,prov'd}}$$ $$Ratio = \frac{(7.2\text{ in})}{(28.5\text{ in})}$$ $$Ratio = 0.25263$$	Status: **PASS** Ratio = **0.253**
	Development length provided along z-direction: $l_{d,prov'd}$ - Development length provided, $$l_{d,prov'd} = \frac{l_z - c_z}{2} - cc_{bot}$$ $$l_{d,prov'd} = \frac{(81\text{ in}) - (18\text{ in})}{2} - (3\text{ in})$$ $$l_{d,prov'd} = 28.5\text{ in}$$	
	Check development length ratio: $Ratio$ $$Ratio = \frac{l_d}{l_{d,prov'd}}$$ $$Ratio = \frac{(7.2\text{ in})}{(28.5\text{ in})}$$ $$Ratio = 0.25263$$	Status: **PASS** Ratio = **0.253**

REFERENCES	CALCULATIONS	RESULTS

SkyCiv Foundation Design

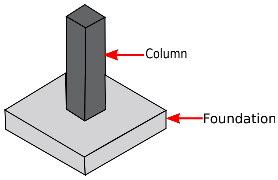
Design Information :

Isolated Foundation
Design code : ACI-318-2014 (American Concrete Institute)
Unit System : Imperial

Project Details :

Project Name : Sunlux
Client : Sunlux
Designer : Travis
Company : MT Solar
Notes :

GEOMETRY:



Foundation:
$l_z = 81\ \mathrm{in}$ - Depth,
$l_x = 81\ \mathrm{in}$ - Width,
$t = 30\ \mathrm{in}$ - Thickness,
Column:
$c_x = 18\ \mathrm{in}$ - Width,
$c_z = 18\ \mathrm{in}$ - Depth,
$h_{column} = 1\ \mathrm{in}$ - Height,
$z_{offset} = 0\ \mathrm{in}$ - Offset Along Z-axis,
$x_{offset} = 0\ \mathrm{in}$ - Offset Along X-axis,

MATERIAL PROPERTIES:

Concrete:
$f'_c = 2500\ \mathrm{psi}$ - Specified compressive strength of concrete,
Steel:
$f_y = 40000\ \mathrm{psi}$ - Specified yield strength for nonprestressed reinforcement,

Densities:
$\gamma_{soil} = 120\ \mathrm{lbf/ft^3}$ - Soil unit weight,
$\gamma_{conc} = 150\ \mathrm{lbf/ft^3}$ - Concrete unit weight,
$\gamma_{water} = 62.4\ \mathrm{lbf/ft^3}$ - Water unit weight,

SOIL PARAMETERS:

$h_{soil} = 0\ \mathrm{in}$ - Soil thickness,
$\theta_{friction} = 33$ - Angle of internal friction,
$\mu_{friction} = 0.45$ - Coefficient of friction,
$q = 0\ \mathrm{ksf}$ - Surcharge area load,
$\sigma_{bearing} = 2\ \mathrm{ksf}$ - Allowable Soil Bearing capacity (gross),

REINFORCEMENTS:

Footing:
Bottom reinforcement (x-direction): 0.375in∅; at 12in spacing
Bottom reinforcement (z-direction): 0.375in∅; at 12in spacing
Column:
Main reinforcement: 3pcs -0.375in∅;
Secondary reinforcement: 0.375in∅; at 12in spacing

MISCELLANEOUS:

Table 20.6.1.3.1 — $cc_{top} = 3\ \mathrm{in}$ - Concrete cover at top,
Table 20.6.1.3.1 — $cc_{bot} = 3\ \mathrm{in}$ - Concrete cover at bottom,
$SW_{factor} = 1$ - Selfweight Factor,

GEOMETRIC VALUES:

A_{ftg} - Area of a square

$$A_{ftg} = l_z\, l_x$$

$$A_{ftg} = (81\ \mathrm{in}) \times (81\ \mathrm{in})$$

$$A_{ftg} = 6561\ \mathrm{in}^2$$

A_{col} - Area of a square

$$A_{col} = c_z\, c_x$$

$$A_{col} = (18\ \mathrm{in}) \times (18\ \mathrm{in})$$

$$A_{col} = 324\ \text{in}^2$$

V_{ftg} - Volume of foundation,

$$V_{ftg} = A_{ftg}\, t$$

$$V_{ftg} = \left(6561\ \text{in}^2\right) \times (30\ \text{in})$$

$$V_{ftg} = 113.91\ \text{ft}^3$$

V_{col} - Volume of column,

$$V_{col} = A_{col}\, h_{column}$$

$$V_{col} = \left(324\ \text{in}^2\right) \times (1\ \text{in})$$

$$V_{col} = 0.1875\ \text{ft}^3$$

SELF WEIGHTS

SW_{col} - Self weight of column,

$$SW_{col} = V_{col}\, \gamma_{conc}\, SW_{factor}$$

$$SW_{col} = \left(0.1875\ \text{ft}^3\right) \times \left(150\ \text{lbf/ft}^3\right) \times (1)$$

$$SW_{col} = 0.028125\ \text{kip}$$

SW_{ftg} - Self weight of foundation,

$$SW_{ftg} = V_{ftg}\, \gamma_{conc}\, SW_{factor}$$

$$SW_{ftg} = \left(113.91\ \text{ft}^3\right) \times \left(150\ \text{lbf/ft}^3\right) \times (1)$$

$$SW_{ftg} = 17.086\ \text{kip}$$

SW_{soil} - Self Weight of Soil,

$$SW_{soil} = \left[\left(A_{ftg}\, h_{soil}\right) - \left(A_{col}\, h_{soil}\right)\right] \gamma_{soil}\, SW_{factor}$$

$$SW_{soil} = \left[\left(\left(6561\ \text{in}^2\right) \times (0\ \text{in})\right) - \left(\left(324\ \text{in}^2\right) \times (0\ \text{in})\right)\right] \times \left(120\ \text{lbf/ft}^3\right) \times (1)$$

$$SW_{soil} = 0\ \text{kip}$$

$SW_{buoyancy} = 0\ \text{kip}$ - Total Buoyant Weight,
SW_{total} - Total weight,
Excluding column weights.

$$SW_{total} = SW_{ftg} + SW_{soil} - SW_{buoyancy}$$

$$SW_{total} = (17.086\ \text{kip}) + (0\ \text{kip}) - (0\ \text{kip})$$

$$SW_{total} = 17.086\ \text{kip}$$

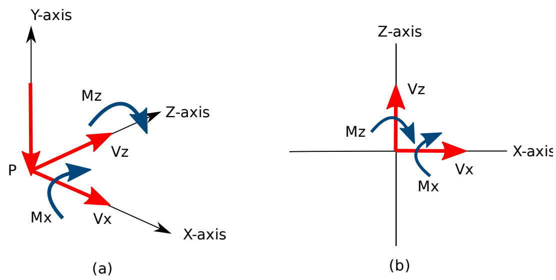
Q - Surcharge load,

$$Q = q\, A_{ftg}$$

$$Q = (0\ \text{ksf}) \times \left(6561\ \text{in}^2\right)$$

$$Q = 0\ \text{kip}$$

LOADS



Local Sign Convention

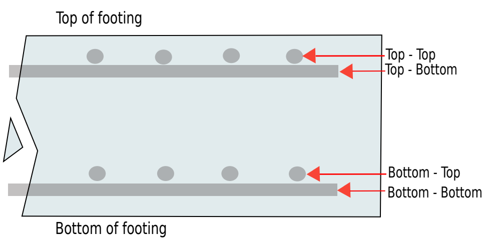
Positive direction (+) in (a) X-Y-Z axis (b) X-Z axis

Tabulation of input loads:

Load Component	Service loads	Ultimate loads
P (kip)	3.959	6.278
V_x (kip)	-4.293	-7.156

V_z (kip)	-0.234	-0.389
M_x (kipft)	-0.390	-0.649
M_z (kipft)	32.541	54.556

EFFECTIVE DEPTH



Designation of rebar positions

d_{bb} - Foundation effective depth at Bottom - Bottom

$$d_{bb} = t - cc_{bot} - \frac{\phi_{bottom,x}}{2.00}$$

$$d_{bb} = (30 \text{ in}) - (3 \text{ in}) - \frac{(0.375 \text{ in})}{2.00}$$

$$d_{bb} = 26.813 \text{ in}$$

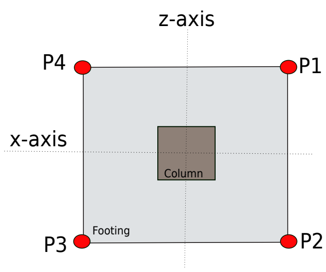
d_{bt} - Foundation effective depth at Bottom - Top

$$d_{bt} = t - cc_{bot} - \phi_{bottom,x} - \frac{\phi_{bottom,z}}{2.00}$$

$$d_{bt} = (30 \text{ in}) - (3 \text{ in}) - (0.375 \text{ in}) - \frac{(0.375 \text{ in})}{2.00}$$

$$d_{bt} = 26.438 \text{ in}$$

SOIL PRESSURE CHECK



Soil Pressure (sls):

Parameters

$SW_{total,sls}$ - Factored total weight,

$$SW_{total,sls} = [DL_{sls}\ (SW_{total} + SW_{col})] + (LL_{sls}\ Q)$$

$$SW_{total,sls} = [(1) \times ((17.086 \text{ kip}) + (0.028125 \text{ kip}))] + ((1) \times (0 \text{ kip}))$$

$$SW_{total,sls} = 17.114 \text{ kip}$$

P_{sls} - Total axial load,

$$P_{sls} = P + SW_{total,sls}$$

$$P_{sls} = (3.9591 \text{ kip}) + (17.114 \text{ kip})$$

$$P_{sls} = 21.073 \text{ kip}$$

$p_{column,sls}$ - Factored column load,

$$p_{column,sls} = P_{sls} - SW_{total,sls,net}$$

$$p_{column,sls} = (21.073 \text{ kip}) - (17.086 \text{ kip})$$

$$p_{column,sls} = 3.9872 \text{ kip}$$

M - Total moment in the x-direction,

$$M = M_{x,sls,1} + [V_{z,sls,1}\ (h_{column} + t)] + (p_{column,sls}\ z_{offset})$$

$$M = (-0.39035 \text{ kipft}) + [(-0.23395 \text{ kip}) \times ((1 \text{ in}) + (30 \text{ in}))] + ((3.9872 \text{ kip}) \times (0 \text{ in}))$$

$$M = -0.99471 \text{ kipft}$$

M_z - Total moment in the z-direction,

M - Total moment in the z-direction,

$$M = M_{z,sls,1} + [V_{x,sls,1}\ (h_{column} + t)] + (p_{column,sls}\ x_{offset})$$

$$M = (32.541\ \text{kipft}) + [(-4.2934\ \text{kip}) \times ((1\ \text{in}) + (30\ \text{in}))] + ((3.9872\ \text{kip}) \times (0\ \text{in}))$$

$$M = 21.45\ \text{kipft}$$

e_x - Eccentricity in the x-direction,

$$e_x = \frac{M}{P_{sls}}$$

$$e_x = \frac{(21.45\ \text{kipft})}{(21.073\ \text{kip})}$$

$$e_x = 1.0179\ \text{ft}$$

e_z - Eccentricity in the z-direction,

$$e_z = \frac{M}{P_{sls}}$$

$$e_z = \frac{(-0.99471\ \text{kipft})}{(21.073\ \text{kip})}$$

$$e_z = -0.047203\ \text{ft}$$

The eccentricity (1.018, -0.047) falls under: **zone C**

S_x - Section modulus,

$$S_x = \frac{l_x\ l_z^2}{6}$$

$$S_x = \frac{(81\ \text{in}) \times (81\ \text{in})^2}{6}$$

$$S_x = 51.258\ \text{ft}^3$$

S_z - Section modulus,

$$S_z = \frac{l_z\ l_x^2}{6}$$

$$S_z = \frac{(81\ \text{in}) \times (81\ \text{in})^2}{6}$$

$$S_z = 51.258\ \text{ft}^3$$

P_1 - Soil pressure at

$$P_1 = \frac{P_{sls}}{A_{ftg}} + \frac{M}{S_z} + \frac{M}{S_x}$$

$$P_1 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} + \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} + \frac{(-0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_1 = 0.86158\ \text{ksf}$$

P_2 - Soil pressure at

$$P_2 = \frac{P_{sls}}{A_{ftg}} + \frac{M}{S_z} - \frac{M}{S_x}$$

$$P_2 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} + \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} - \frac{(-0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_2 = 0.90039\ \text{ksf}$$

P_3 - Soil pressure at

$$P_3 = \frac{P_{sls}}{A_{ftg}} - \frac{M}{S_z} - \frac{M}{S_x}$$

$$P_3 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} - \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} - \frac{(-0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_3 = 0.063443\ \text{ksf}$$

P_4 - Soil pressure at

$$P_4 = \frac{P_{sls}}{A_{ftg}} - \frac{M}{S_z} + \frac{M}{S_x}$$

$$P_4 = \frac{(21.073\ \text{kip})}{(6561\ \text{in}^2)} - \frac{(21.45\ \text{kipft})}{(51.258\ \text{ft}^3)} + \frac{(-0.99471\ \text{kipft})}{(51.258\ \text{ft}^3)}$$

$$P_4 = 0.024631\ \text{ksf}$$

q_{sls} - Maximum unfactored soil pressure,

$$q_{sls} = MAX\left[P_1, P_2, P_3, P_4\right]$$

$$q_{sls} = MAX\left[(0.86158 \text{ ksf}), (0.90039 \text{ ksf}), (0.063443 \text{ ksf}), (0.024631 \text{ ksf})\right]$$

$$q_{sls} = 0.90039 \text{ kip/ft}^2$$

Soil Pressure (uls):

Parameters

$SW_{total,uls}$ - Factored total weight,

$$SW_{total,uls} = \left[DL_{uls}\ (SW_{total} + SW_{col})\right] + (LL_{uls}\ Q)$$

$$SW_{total,uls} = \left[(1.2) \times ((17.086 \text{ kip}) + (0.028125 \text{ kip}))\right] + ((1.6) \times (0 \text{ kip}))$$

$$SW_{total,uls} = 20.537 \text{ kip}$$

P_{uls} - Total axial load,

$$P_{uls} = P + SW_{total,uls}$$

$$P_{uls} = (6.2778 \text{ kip}) + (20.537 \text{ kip})$$

$$P_{uls} = 26.815 \text{ kip}$$

$p_{column,uls}$ - Factored column load,

$$p_{column,uls} = P_{uls} - SW_{total,uls,net}$$

$$p_{column,uls} = (26.815 \text{ kip}) - (20.503 \text{ kip})$$

$$p_{column,uls} = 6.3115 \text{ kip}$$

M - Total moment in the x-direction,

$$M = M_{x,uls,1} + \left[V_{z,uls,1}\ (h_{column} + t)\right] + (p_{column,uls}\ z_{offset})$$

$$M = (-0.64935 \text{ kipft}) + \left[(-0.38917 \text{ kip}) \times ((1 \text{ in}) + (30 \text{ in}))\right] + ((6.3115 \text{ kip}) \times (0 \text{ in}))$$

$$M = -1.6547 \text{ kipft}$$

M - Total moment in the z-direction,

$$M = M_{z,uls,1} + \left[V_{x,uls,1}\ (h_{column} + t)\right] + (p_{column,uls}\ x_{offset})$$

$$M = (54.556 \text{ kipft}) + \left[(-7.1556 \text{ kip}) \times ((1 \text{ in}) + (30 \text{ in}))\right] + ((6.3115 \text{ kip}) \times (0 \text{ in}))$$

$$M = 36.071 \text{ kipft}$$

e_x - Eccentricity in the x-direction,

$$e_x = \frac{M}{P_{uls}}$$

$$e_x = \frac{(36.071 \text{ kipft})}{(26.815 \text{ kip})}$$

$$e_x = 1.3452 \text{ ft}$$

e_z - Eccentricity in the z-direction,

$$e_z = \frac{M}{P_{uls}}$$

$$e_z = \frac{(-1.6547 \text{ kipft})}{(26.815 \text{ kip})}$$

$$e_z = -0.061709 \text{ ft}$$

The eccentricity (1.345, -0.062) falls under: **zone E**

P_2

$$P_2 = \frac{P_{uls}}{3\, l_x\, l_z} \left(2 - \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \left(1 + \frac{6\, e_z}{l_z} + \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \frac{1}{\frac{1}{2} - \frac{e_x}{l_x}}$$

$$P_2 = \frac{(26.815)}{3 \times (81) \times (81)} \times \left(2 - \sqrt{1 - \frac{12 \times (-0.061709)^2}{(81)^2}}\right) \times \left(1 + \frac{6 \times (-0.061709)}{(81)} + \sqrt{1 - \frac{12 \times (-0.061709)^2}{(81)^2}}\right) \times \frac{1}{\frac{1}{2} - \frac{(1.3452)}{(81)}}$$

$$P_2 = 1.3409 \text{ kip/ft}^2$$

P_1

$$P_1 = \frac{P_{uls}}{3\, l_x\, l_z} \left(2 - \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \left(1 - \frac{6\, e_z}{l_z} + \sqrt{1 - \frac{12\, e_z^2}{l_z^2}}\right) \frac{1}{\frac{1}{2} - \frac{e_x}{l_x}}$$

$$P_1 = \frac{(26.815)}{3 \times (81) \times (81)} \times \left(2 - \sqrt{1 - \frac{12 \times (-0.061709)^2}{(81)^2}}\right) \times \left(1 - \frac{6 \times (-0.061709)}{(81)} + \sqrt{1 - \frac{12 \times (-0.061709)^2}{(81)^2}}\right) \times \frac{1}{\frac{1}{2} - \frac{(1.3452)}{(81)}}$$

$$P_1 = 1.2693\ \mathrm{kip/ft^2}$$

$P_3 = 0\ \mathrm{kip/ft^2}$ - Soil pressure at

$P_4 = 0\ \mathrm{kip/ft^2}$ - Soil pressure at

q_{uls} - Maximum factored soil pressure,

$$q_{uls} = MAX\left[P_1, P_2, P_3, P_4\right]$$

$$q_{uls} = MAX\left[\left(1.2693\ \mathrm{kip/ft^2}\right), \left(1.3409\ \mathrm{kip/ft^2}\right), \left(0\ \mathrm{kip/ft^2}\right), \left(0\ \mathrm{kip/ft^2}\right)\right]$$

$$q_{uls} = 1.3409\ \mathrm{kip/ft^2}$$

Summary:

Corner Pressure	p1	p2	p3	p4
Service	0.862	0.900	0.063	0.025
Ultimate	1.269	1.341	0.000	0.000

Check soil bearing capacity ratio:

$Ratio$

$$Ratio = \frac{q_{sls}}{\sigma_{bearing}}$$

$$Ratio = \frac{\left(0.90039\ \mathrm{kip/ft^2}\right)}{\left(2\ \mathrm{ksf}\right)}$$

$$Ratio = 0.4502$$

Status: **PASS**
Ratio = **0.450**

UPLIFT CHECK

Uplift stability considers axial loads only

Governing SLS axial force = 21.073 kip

∴ Uplift does not exist!

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

SLIDING CHECK

$\phi_{friction}$ - Angle of internal friction (rad),

$$\phi_{friction} = \theta_{friction}\ \frac{\pi}{180}$$

$$\phi_{friction} = (33) \times \frac{\pi}{180}$$

$$\phi_{friction} = 0.57596$$

k_p - Passive Coefficient,

$$k_p = \frac{1 + \sin\phi_{friction}}{1 - \sin\phi_{friction}}$$

$$k_p = \frac{1 + \sin(0.57596)}{1 - \sin(0.57596)}$$

$$k_p = 3.3921$$

P_{mid} - Pressure at mid depth,

$$P_{mid} = k_p\ \gamma_{soil}\ (h_{soil} + \frac{t}{2})$$

$$P_{mid} = (3.3921) \times \left(120\ \mathrm{lbf/ft^3}\right) \times ((0\ \mathrm{in}) + \frac{(30\ \mathrm{in})}{2})$$

$$P_{mid} = 0.50882\ \mathrm{ksf}$$

Sliding check along the x-direction:

$F_{p,x}$ - Passive Force,

$$F_{p,x} = P_{mid}\ t\ l_z$$

$$F_{p,x} = (0.50882\ \mathrm{ksf}) \times (30\ \mathrm{in}) \times (81\ \mathrm{in})$$

$$F_{p,x} = 8.5863\ \mathrm{kip}$$

F_f - Friction Force,

$$F_f = \mu_{friction}\ P_{sls}$$

$$F_f = (0.45) \times (21.073\ \mathrm{kip})$$

$$F_f = 9.4829 \text{ kip}$$

FOS_{actual} - Actual Factor of Safety for Sliding,

$$FOS_{actual} = \frac{F_{p,x} + F_f}{V_{x,sls,1}}$$

$$FOS_{actual} = \frac{(8.5863 \text{ kip}) + (9.4829 \text{ kip})}{(-4.2934 \text{ kip})}$$

$$FOS_{actual} = -4.2086$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(1.5)}{(-4.2086)}$$

$$Ratio = 0.35641$$

Status: **PASS**
Ratio = **0.356**

Sliding check along the z-direction:

$F_{p,z}$ - Passive Force,

$$F_{p,z} = P_{mid}\, t\, l_x$$

$$F_{p,z} = (0.50882 \text{ ksf}) \times (30 \text{ in}) \times (81 \text{ in})$$

$$F_{p,z} = 8.5863 \text{ kip}$$

F_f - Friction Force,

$$F_f = \mu_{friction}\, P_{sls}$$

$$F_f = (0.45) \times (21.073 \text{ kip})$$

$$F_f = 9.4829 \text{ kip}$$

FOS_{actual} - Actual Factor of Safety for Sliding,

$$FOS_{actual} = \frac{F_{p,z} + F_f}{V_{z,sls,1}}$$

$$FOS_{actual} = \frac{(8.5863 \text{ kip}) + (9.4829 \text{ kip})}{(-0.23395 \text{ kip})}$$

$$FOS_{actual} = -77.236$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(1.5)}{(-77.236)}$$

$$Ratio = 0.019421$$

Status: **PASS**
Ratio = **0.019**

OVERTURNING CHECK

Overturning along the x-direction:

FOS_{actual} - Actual Factor of Safety for Overturning,

$$FOS_{actual} = \frac{(\frac{l_x}{2})}{e_x}$$

$$FOS_{actual} = \frac{(\frac{(81 \text{ in})}{2})}{(1.0179 \text{ ft})}$$

$$FOS_{actual} = 3.3157$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(2)}{(3.3157)}$$

$$Ratio = 0.60319$$

Status: **PASS**
Ratio = **0.603**

Overturning along the z-direction:

FOS_{actual} - Actual Factor of Safety for Overturning,

$$FOS_{actual} = \frac{(\frac{l_z}{2})}{e_z}$$

$$FOS_{actual} = \frac{(\frac{(81\ \text{in})}{2})}{(-0.047203\ \text{ft})}$$

$$FOS_{actual} = 71.5$$

$Ratio$

$$Ratio = \frac{FOS_{allowable}}{FOS_{actual}}$$

$$Ratio = \frac{(2)}{(71.5)}$$

$$Ratio = 0.027972$$

Status: **PASS**
Ratio = **0.028**

ONE WAY SHEAR CHECK

Table 21.2.2 $\theta_{shear} = 0.75$ - Shear reduction factor,

d - Critical depth,

$$d = \frac{d_{bb} + d_{bt}}{2}$$

$$d = \frac{(26.813\ \text{in}) + (26.438\ \text{in})}{2}$$

$$d = 26.625\ \text{in}$$

Critical section along the x-direction:

20.6.1.3.1 $V_{demand,x}$ - Shear demand,

$$V_{demand,x} = q_{uls}\left(\frac{l_z}{2} + |z_{offset}| - \frac{c_z}{2} - d\right) l_x$$

$$V_{demand,x} = \left(1.3409\ \text{kip/ft}^2\right) \times \left(\frac{(81\ \text{in})}{2} + |\,(0\ \text{in})\,| - \frac{(18\ \text{in})}{2} - (26.625\ \text{in})\right) \times (81\ \text{in})$$

$$V_{demand,x} = 3.6769\ \text{kip}$$

$V_{capacity,x}$ - Shear Capacity

$$V_{capacity,x} = \theta_{shear}\ 2\ \lambda \sqrt{f_c'}\ l_z\ d$$

$$V_{capacity,x} = (0.75) \times 2 \times (1) \times \sqrt{(2500\ \text{psi})} \times (81\ \text{in}) \times (26.625\ \text{in})$$

$$V_{capacity,x} = 161750\ \text{lbf}$$

Critical section along the z-direction:

20.6.1.3.1 $V_{demand,z}$ - Shear demand,

$$V_{demand,z} = q_{uls}\left(\frac{l_x}{2} + |x_{offset}| - \frac{c_x}{2} - d\right) l_z$$

$$V_{demand,z} = \left(1.3409\ \text{kip/ft}^2\right) \times \left(\frac{(81\ \text{in})}{2} + |\,(0\ \text{in})\,| - \frac{(18\ \text{in})}{2} - (26.625\ \text{in})\right) \times (81\ \text{in})$$

$$V_{demand,z} = 3.6769\ \text{kip}$$

$V_{capacity,z}$ - Shear Capacity

$$V_{capacity,z} = \theta_{shear}\ 2\ \lambda \sqrt{f_c'}\ l_x\ d$$

$$V_{capacity,z} = (0.75) \times 2 \times (1) \times \sqrt{(2500\ \text{psi})} \times (81\ \text{in}) \times (26.625\ \text{in})$$

$$V_{capacity,z} = 161750\ \text{lbf}$$

Governing shear capacity and demand:

V_{demand} - Governing shear demand,

$$V_{demand} = MAX\left[V_{demand,x}, V_{demand,z}\right]$$

$$V_{demand} = MAX\left[(3.6769\ \text{kip}), (3.6769\ \text{kip})\right]$$

$$V_{demand} = 3.6769\ \text{kip}$$

$V_{capacity}$ - Governing shear capacity,

$$V_{capacity} = MIN\left[V_{capacity,x}, V_{capacity,z}\right]$$

$$V_{capacity} = MIN\left[(161750\ \text{lbf}), (161750\ \text{lbf})\right]$$

$$V_{capacity} = 161.75\ \text{kip}$$

Check capacity ratio:
$Ratio$

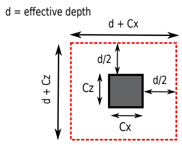
$$Ratio = \frac{V_{demand}}{V_{capacity}}$$

$$Ratio = \frac{(3.6769\ \text{kip})}{(161.75\ \text{kip})}$$

$$Ratio = 0.022733$$

Status: **PASS**
Ratio = **0.023**

TWO WAY SHEAR CHECK



PLAN

Parameters:

22.6.5.3 — $\alpha_s = 40$ - Column position factor (interior)

$\lambda = 1$ - Concrete weight type factor,

22.6.4.1 — b_o - Critical shear perimeter,

$$b_o = \left[(d + c_x) + (d + c_z)\right]\ 2$$

$$b_o = \left[((26.625\ \text{in}) + (18\ \text{in})) + ((26.625\ \text{in}) + (18\ \text{in}))\right] \times 2$$

$$b_o = 178.5\ \text{in}$$

$A_{punching}$ - Punching shear area,

$$A_{punching} = (d + c_x)\ (d + c_z)$$

$$A_{punching} = ((26.625\ \text{in}) + (18\ \text{in})) \times ((26.625\ \text{in}) + (18\ \text{in}))$$

$$A_{punching} = 1991.4\ \text{in}^2$$

$V_{tw,demand}$ - Punching shear demand

$$V_{tw,demand} = (A_{ftg} - A_{punching})\ q_{uls}$$

$$V_{tw,demand} = \left(\left(6561\ \text{in}^2\right) - \left(1991.4\ \text{in}^2\right)\right) \times \left(1.3409\ \text{kip/ft}^2\right)$$

$$V_{tw,demand} = 42.55\ \text{kip}$$

22.6.5.2 (a) — $V_{capacity,a}$ - Two-way shear capacity (a)

$$V_{capacity,a} = 4\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,a} = 4 \times (1) \times \sqrt{(2500\ \text{psi})} \times (178.5\ \text{in}) \times (26.625\ \text{in})$$

$$V_{capacity,a} = 950510\ \text{lbf}$$

β - Ratio of the long side to short side of column,

$$\beta = \frac{MAX\,(c_x, c_z)}{MIN\,(c_x, c_z)}$$

$$\beta = \frac{MAX\,((18\ \text{in}), (18\ \text{in}))}{MIN\,((18\ \text{in}), (18\ \text{in}))}$$

$$\beta = 1$$

22.6.5.2 (b) — $V_{capacity,b}$ - Two-way shear capacity {b}

$$V_{capacity,b} = \left(2 + \frac{4}{\beta}\right)\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,b} = \left(2 + \frac{4}{(1)}\right) \times (1) \times \sqrt{(2500\ \text{psi})} \times (178.5\ \text{in}) \times (26.625\ \text{in})$$

$$V_{capacity,b} = 1.4258{\times}10^{6}\ \text{lbf}$$

22.6.5.2 (c) — $V_{capacity,c}$ - Two-way shear capacity (c)

$$V_{capacity,c} = \left(2 + \frac{\alpha_s\ d}{b_o}\right)\ \lambda\ \sqrt{f_c'}\ b_o\ d$$

$$V_{capacity,c} = \left(2 + \frac{(40) \times (26.625 \text{ in})}{(178.5 \text{ in})}\right) \times (1) \times \sqrt{(2500 \text{ psi})} \times (178.5 \text{ in}) \times (26.625 \text{ in})$$

$$V_{capacity,c} = 1.893{\times}10^{6} \text{ lbf}$$

$V_{tw,capacity}$ - Governing shear capacity,

$$V_{tw,capacity} = MIN\left[V_{capacity,a}, V_{capacity,b}, V_{capacity,c}\right]$$

$$V_{tw,capacity} = MIN\left[(950510 \text{ lbf}), (1.4258{\times}10^{6} \text{ lbf}), (1.893{\times}10^{6} \text{ lbf})\right]$$

$$V_{tw,capacity} = 950.51 \text{ kip}$$

Check capacity ratio:

$Ratio$

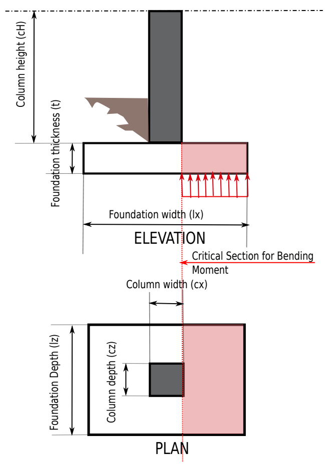
$$Ratio = \frac{V_{tw,demand}}{V_{tw,capacity}}$$

$$Ratio = \frac{(42.55 \text{ kip})}{(950.51 \text{ kip})}$$

$$Ratio = 0.044766$$

Status: **PASS**
Ratio = **0.045**

FLEXURE CHECK



Flexural check along the x-direction:

$M_{demand,x}$ - Moment demand

$$M_{demand,x} = q_{uls}\left(\frac{l_x - c_x}{2} + |x_{offset}|\right) l_z \frac{\left(\frac{l_x - c_x}{2} + |x_{offset}|\right)}{2}$$

$$M_{demand,x} = \left(1.3409 \text{ kip/ft}^2\right) \times \left(\frac{(81 \text{ in}) - (18 \text{ in})}{2} + |(0 \text{ in})|\right) \times (81 \text{ in}) \times \frac{\left(\frac{(81 \text{ in}) - (18 \text{ in})}{2} + |(0 \text{ in})|\right)}{2}$$

$$M_{demand,x} = 31.183 \text{ kipft}$$

n - No. of reinforcement

$$n = \left[\frac{l_z - (2\, cc_{bot}) - \phi_{bottom,x}}{s_{bottom,x}}\right] + 1$$

$$n = \left[\frac{(81 \text{ in}) - (2 \times (3 \text{ in})) - (0.375 \text{ in})}{(12 \text{ in})}\right] + 1$$

$$n = 8$$

A_g - Area gross of the foundation

$$A_g = t\, l_z$$

$$A_g = (30 \text{ in}) \times (81 \text{ in})$$

$$A_g = 2430 \text{ in}^2$$

Table 8.6.1.1 $A_{s,min}$ - Minimum area of reinforcement

$$A_{s,min} = 0.0020\, A_g$$

$$A_{s,min} = 0.0020 \times \left(2430 \text{ in}^2\right)$$

$$A_{s,min} = 4.86 \text{ in}^2$$

A_{rebar} - Reinforcement area

$$A_{rebar} = \frac{\pi}{4} \left(\phi_{bottom,x}\right)^2$$

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

$$A_{rebar} = 0.11045 \text{ in}^2$$

$A_{s,prov}$ - Total reinforcement area

$$A_{s,prov} = n \, \frac{\pi \, \phi_{bottom,x}^2}{4}$$

$$A_{s,prov} = (8) \times \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_{s,prov} = 0.88357 \text{ in}^2$$

Check reinforcement ratio:

$Ratio$

$$Ratio = \frac{A_{s,min}}{A_{s,prov}}$$

$$Ratio = \frac{\left(4.86 \text{ in}^2\right)}{\left(0.88357 \text{ in}^2\right)}$$

$$Ratio = 5.5004$$

Status: **FAIL**
Ratio = **5.500**

Since 2500 psi $\leq f'_c \leq$ 4000 psi:

Table 22.2.2.4.3 $\beta_1 = 0.85$ - Equivalent rectangular concrete stress distribution

a - Depth of equivalent rectangular stress block,

$$a = \frac{A_{s,prov} \, f_y}{0.85 \, f'_c \, l_z}$$

$$a = \frac{\left(0.88357 \text{ in}^2\right) \times (40000 \text{ psi})}{0.85 \times (2500 \text{ psi}) \times (81 \text{ in})}$$

$$a = 0.20533 \text{ in}$$

22.2.2.4.2 c - Distance from the extreme compression fiber to neutral axis,

$$c = \frac{a}{\beta_1}$$

$$c = \frac{(0.20533 \text{ in})}{(0.85)}$$

$$c = 0.24157 \text{ in}$$

21.2.2 $\varepsilon_c = 0.003$ - Compressive strain,

21.2.2 ε_t - Tensile strain in the reinforcement,

$$\varepsilon_t = \frac{\varepsilon_c}{c} \left(d - c\right)$$

$$\varepsilon_t = \frac{(0.003)}{(0.24157 \text{ in})} \times ((26.625 \text{ in}) - (0.24157 \text{ in}))$$

$$\varepsilon_t = 0.32765$$

Since $\varepsilon_t \geq 0.005$ (Tension-controlled):

Table 21.2.2 $\phi = 0.9$ - Strength reduction factor:,

$M_{capacity,x}$ - Moment Capacity

$$M_{capacity,x} = \phi \, A_{s,prov} \, f_y \left(d - \frac{a}{2}\right)$$

$$M_{capacity,x} = (0.9) \times \left(0.88357 \text{ in}^2\right) \times (40000 \text{ psi}) \times \left((26.625 \text{ in}) - \frac{(0.20533 \text{ in})}{2}\right)$$

$$M_{capacity,x} = 70.303 \text{ kipft}$$

Flexural check along the z-direction:

$M_{demand,z}$ - Moment demand

$$M_{demand,z} = q_{uls} \left(\frac{l_z - c_z}{2} + |z_{offset}|\right) l_x$$

$$\left(\frac{l_z - c_z}{2} + |z_{offset}|\right)$$

$$\frac{\left(\quad \right)}{2}$$

$$M_{demand,z} = \left(1.3409 \text{ kip/ft}^2\right) \times \left(\frac{(81 \text{ in}) - (18 \text{ in})}{2} + |(0 \text{ in})|\right) \times (81 \text{ in}) \times \frac{\left(\frac{(81 \text{ in})-(18 \text{ in})}{2} + |(0 \text{ in})|\right)}{2}$$

$$M_{demand,z} = 31.183 \text{ kipft}$$

n - No. of reinforcement

$$n = \left[\frac{l_x - (2\, cc_{bot}) - \phi_{bottom,z}}{s_{bottom,z}}\right] + 1$$

$$n = \left[\frac{(81 \text{ in}) - (2 \times (3 \text{ in})) - (0.375 \text{ in})}{(12 \text{ in})}\right] + 1$$

$$n = 8$$

A_g - Area gross of the foundation

$$A_g = t\, l_x$$

$$A_g = (30 \text{ in}) \times (81 \text{ in})$$

$$A_g = 2430 \text{ in}^2$$

Table 8.6.1.1 $A_{s,min}$ - Minimum area of reinforcement

$$A_{s,min} = 0.0020\, A_g$$

$$A_{s,min} = 0.0020 \times \left(2430 \text{ in}^2\right)$$

$$A_{s,min} = 4.86 \text{ in}^2$$

A_{rebar} - Reinforcement area

$$A_{rebar} = \frac{\pi}{4}\,(\phi_{bottom,z})^2$$

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

$$A_{rebar} = 0.11045 \text{ in}^2$$

$A_{s,prov}$ - Total reinforcement area

$$A_{s,prov} = n\, \frac{\pi\, \phi_{bottom,z}^2}{4}$$

$$A_{s,prov} = (8) \times \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_{s,prov} = 0.88357 \text{ in}^2$$

Check reinforcement ratio:

$Ratio$

$$Ratio = \frac{A_{s,min}}{A_{s,prov}}$$

$$Ratio = \frac{\left(4.86 \text{ in}^2\right)}{\left(0.88357 \text{ in}^2\right)}$$

$$Ratio = 5.5004$$

Status: **FAIL**
Ratio = **5.500**

Since 2500 psi $\leq f'_c \leq$ 4000 psi:

Table 22.2.2.4.3 $\beta_1 = 0.85$ - Equivalent rectangular concrete stress distribution

a - Depth of equivalent rectangular stress block,

$$a = \frac{A_{s,prov}\, f_y}{0.85\, f'_c\, l_x}$$

$$a = \frac{\left(0.88357 \text{ in}^2\right) \times (40000 \text{ psi})}{0.85 \times (2500 \text{ psi}) \times (81 \text{ in})}$$

$$a = 0.20533 \text{ in}$$

22.2.2.4.2 c - Distance from the extreme compression fiber to neutral axis,

$$c = \frac{a}{\beta_1}$$

$$c = \frac{(0.20533 \text{ in})}{(0.85)}$$

$$c = 0.24157\ \text{in}$$

21.2.2 $\varepsilon_c = 0.003$ - Compressive strain,

21.2.2 ε_t - Tensile strain in the reinforcement,

$$\varepsilon_t = \frac{\varepsilon_c}{c}(d - c)$$

$$\varepsilon_t = \frac{(0.003)}{(0.24157\ \text{in})} \times ((26.625\ \text{in}) - (0.24157\ \text{in}))$$

$$\varepsilon_t = 0.32765$$

Since $\varepsilon_t \geq 0.005$ (Tension-controlled):

Table 21.2.2 $\phi = 0.9$ - Strength reduction factor:,

$M_{capacity,z}$ - Moment Capacity

$$M_{capacity,z} = \phi\ A_{s,prov}\ f_y\ \left(d - \frac{a}{2}\right)$$

$$M_{capacity,z} = (0.9) \times \left(0.88357\ \text{in}^2\right) \times (40000\ \text{psi}) \times \left((26.625\ \text{in}) - \frac{(0.20533\ \text{in})}{2}\right)$$

$$M_{capacity,z} = 70.303\ \text{kipft}$$

Governing moment capacity and demand:

M_{demand} - Governing moment demand,

$$M_{demand} = MAX\left[M_{demand,x}, M_{demand,z}\right]$$

$$M_{demand} = MAX\left[(31.183\ \text{kipft}), (31.183\ \text{kipft})\right]$$

$$M_{demand} = 31.183\ \text{kipft}$$

$M_{capacity}$ - Governing moment capacity,

$$M_{capacity} = MIN\left[M_{capacity,x}, M_{capacity,z}\right]$$

$$M_{capacity} = MIN\left[(70.303\ \text{kipft}), (70.303\ \text{kipft})\right]$$

$$M_{capacity} = 70.303\ \text{kipft}$$

Check capacity ratio:

$Ratio$

$$Ratio = \frac{M_{demand}}{M_{capacity}}$$

$$Ratio = \frac{(31.183\ \text{kipft})}{(70.303\ \text{kipft})}$$

$$Ratio = 0.44355$$

Status: **PASS**
Ratio = **0.444**

LOAD TRANSFER OF COLUMN FORCES TO THE BASE

Nominal bearing strength:

ϕB_n

$$\phi B_n = MIN\left[\left(\phi \times \sqrt{\frac{A_1}{A_2}} \times 0.85 \times f_c' \times A_1\right), \left(\phi \times 2\left(\times 0.85 \times f_c' \times A_1\right)\right)\right]$$

Where:

$A_1 = 324\ \text{in}^2$ - Bearing area of column,

$A_2 = 6561\ \text{in}^2$ - Footing area,

21.2.1(d) $\phi = 0.65$ - Bearing strength reduction factor,

Table 22.8.3.2 ϕB_n

$$\phi B_n = MIN\left[\left(\phi\ \sqrt{\frac{A_1}{A_2}}\ 0.85\ f_c'\ A_1\right), \left(\phi\ 2\left(\ 0.85\ f_c'\ A_1\right)\right)\right]$$

$$\phi B_n = MIN\left[\left((0.65) \times \sqrt{\frac{(324\ \text{in}^2)}{(6561\ \text{in}^2)}} \times 0.85 \times (2500\ \text{psi}) \times \left(324\ \text{in}^2\right)\right), \left((0.65) \times 2\left(\times 0.85 \times (2500\ \text{psi}) \times \left(324\ \text{in}^2\right)\right)\right)\right]$$

$$\phi B_n = 895.05\ \text{kip}$$

Check capacity ratio:

$Ratio$

	$$Ratio = \frac{P_{axial}}{\phi B_n}$$ $$Ratio = \frac{(26.815\ \mathrm{kip})}{(895.05\ \mathrm{kip})}$$ $$Ratio = 0.02996$$	Status: **PASS** Ratio = **0.030**
	DEVELOPMENT LENGTH **Compression development length (column):** $\psi_r = 1$ - Confining reinforcement factor, Compression development length shall be the greater of the following $l_{dc,1}$, $l_{dc,2}$, 8 in :	
25.4.9.1	$l_{dc,1}$ $$l_{dc,1} = \phi_{column}\ \frac{f_y\ \psi_r}{50\ \lambda\ \sqrt{f_c'}}$$ $$l_{dc,1} = (0.375\ \mathrm{in}) \times \frac{(40000\ \mathrm{psi}) \times (1)}{50 \times (1) \times \sqrt{(2500\ \mathrm{psi})}}$$ $$l_{dc,1} = 6\ \mathrm{in}$$	
25.4.9.1	$l_{dc,2}$ $$l_{dc,2} = 0.0003\ f_y\ \psi_r\ \phi_{column}$$ $$l_{dc,2} = 0.0003 \times (40000\ \mathrm{psi}) \times (1) \times (0.375\ \mathrm{in})$$ $$l_{dc,2} = 4.5\ \mathrm{in}$$	
25.4.9.1	l_{dc} $$l_{dc} = MAX\left[l_{dc,1}, l_{dc,2}, 8\ \mathrm{in}\right]$$ $$l_{dc} = MAX\left[(6\ \mathrm{in}), (4.5\ \mathrm{in}), (8\ \mathrm{in})\right]$$ $$l_{dc} = 8\ \mathrm{in}$$	
Table 25.3.2	r - Minimum bend diameter, $$r = 4\ \phi_{column}$$ $$r = 4 \times (0.375\ \mathrm{in})$$ $$r = 1.5\ \mathrm{in}$$	
	$h_{req'd}$ - Required depth, $$h_{req'd} = cc_{bot} + \phi_{bottom,x} + \phi_{bottom,z} + \phi_{column} + r + l_{dc}$$ $$h_{req'd} = (3\ \mathrm{in}) + (0.375\ \mathrm{in}) + (0.375\ \mathrm{in}) + (0.375\ \mathrm{in}) + (1.5\ \mathrm{in}) + (8\ \mathrm{in})$$ $$h_{req'd} = 13.625\ \mathrm{in}$$	
	Check footing depth provided: $t = 30\ \mathrm{in}$ - Thickness, $Ratio$ $$Ratio = \frac{h_{req'd}}{t}$$ $$Ratio = \frac{(13.625\ \mathrm{in})}{(30\ \mathrm{in})}$$ $$Ratio = 0.45417$$	Status: **PASS** Ratio = **0.454**
	Flexural reinforcement bar development: l_d - Bar development length, $$l_d = \left[\frac{3 \times f_y}{40 \times \lambda \times \sqrt{f_c'}} \times \frac{\psi_t \times \psi_e \times \psi_s}{\frac{c_b + K_{tr}}{d_b}}\right] \times d_b$$ Parameters:	
Table 25.4.2.4	$\psi_t = 1$ - Casting position factor,	
Table 25.4.2.4	$\psi_e = 1$ - Coating factor,	
Table 25.4.2.4	$\psi_s = 0.8$ - Bar size factor	
25.4.2.3 (b)	$K_{tr} = 0\ \mathrm{in}$ - Transverse reinforcement index,	
25.4.2.3	c_b $$c_b = MIN\left[\left(cc_{bot} + \frac{\phi_{bottom,x}}{2}\right), \left(cc_{bot} + \frac{\phi_{bottom,z}}{2}\right), \frac{s_{bottom,x}}{2}, \frac{s_{bottom,z}}{2}\right]$$	

$$c_b = MIN\left[\left((3\ \text{in}) + \frac{(0.375\ \text{in})}{2}\right), \left((3\ \text{in}) + \frac{(0.375\ \text{in})}{2}\right), \frac{(12\ \text{in})}{2}, \frac{(12\ \text{in})}{2}\right]$$

$$c_b = 3.1875\ \text{in}$$

25.4.2.3	$\frac{c_b+K_{tr}}{d_b}$ - Confinement term	

$$\frac{c_b + K_{tr}}{d_b} = MIN\left[\frac{c_b + K_{tr}}{\phi_{column}}, 2.5\right]$$

$$\frac{c_b + K_{tr}}{d_b} = MIN\left[\frac{(3.1875\ \text{in}) + (0\ \text{in})}{(0.375\ \text{in})}, 2.5\right]$$

$$\frac{c_b + K_{tr}}{d_b} = 2.5$$

25.4.2.3 l_d - Bar development length,

$$l_d = \left[\frac{3\ f_y}{40\ \lambda\ \sqrt{f'_c}}\ \frac{\psi_t\ \psi_e\ \psi_s}{\frac{c_b+K_{tr}}{d_b}}\right] \phi_{column}$$

$$l_d = \left[\frac{3 \times (40000\ \text{psi})}{40 \times (1) \times \sqrt{(2500\ \text{psi})}} \times \frac{(1) \times (1) \times (0.8)}{(2.5)}\right] \times (0.375\ \text{in})$$

$$l_d = 7.2\ \text{in}$$

Development length provided along x-direction:

$l_{d,prov'd}$ - Development length provided,

$$l_{d,prov'd} = \frac{l_x - c_x}{2} - cc_{bot}$$

$$l_{d,prov'd} = \frac{(81\ \text{in}) - (18\ \text{in})}{2} - (3\ \text{in})$$

$$l_{d,prov'd} = 28.5\ \text{in}$$

Check development length ratio:

$Ratio$

$$Ratio = \frac{l_d}{l_{d,prov'd}}$$

$$Ratio = \frac{(7.2\ \text{in})}{(28.5\ \text{in})}$$

$$Ratio = 0.25263$$

Status: **PASS**
Ratio = **0.253**

Development length provided along z-direction:

$l_{d,prov'd}$ - Development length provided,

$$l_{d,prov'd} = \frac{l_z - c_z}{2} - cc_{bot}$$

$$l_{d,prov'd} = \frac{(81\ \text{in}) - (18\ \text{in})}{2} - (3\ \text{in})$$

$$l_{d,prov'd} = 28.5\ \text{in}$$

Check development length ratio:

$Ratio$

$$Ratio = \frac{l_d}{l_{d,prov'd}}$$

$$Ratio = \frac{(7.2\ \text{in})}{(28.5\ \text{in})}$$

$$Ratio = 0.25263$$

Status: **PASS**
Ratio = **0.253**