Project Details

Project Name: Uthmann Family Feeders - 3x8 - V1Jb

Location: 5200 N County Rd 19, Fort Collins, CO 80524, USA

Unique ID: 3P-19.75-8TOP-HD-72-L-3Hx8W-K0K3

Dealer: ______________________________

Date: Fri Feb 28 2025

Number of Modules: 24

Number of Poles: 3

Date Sold:

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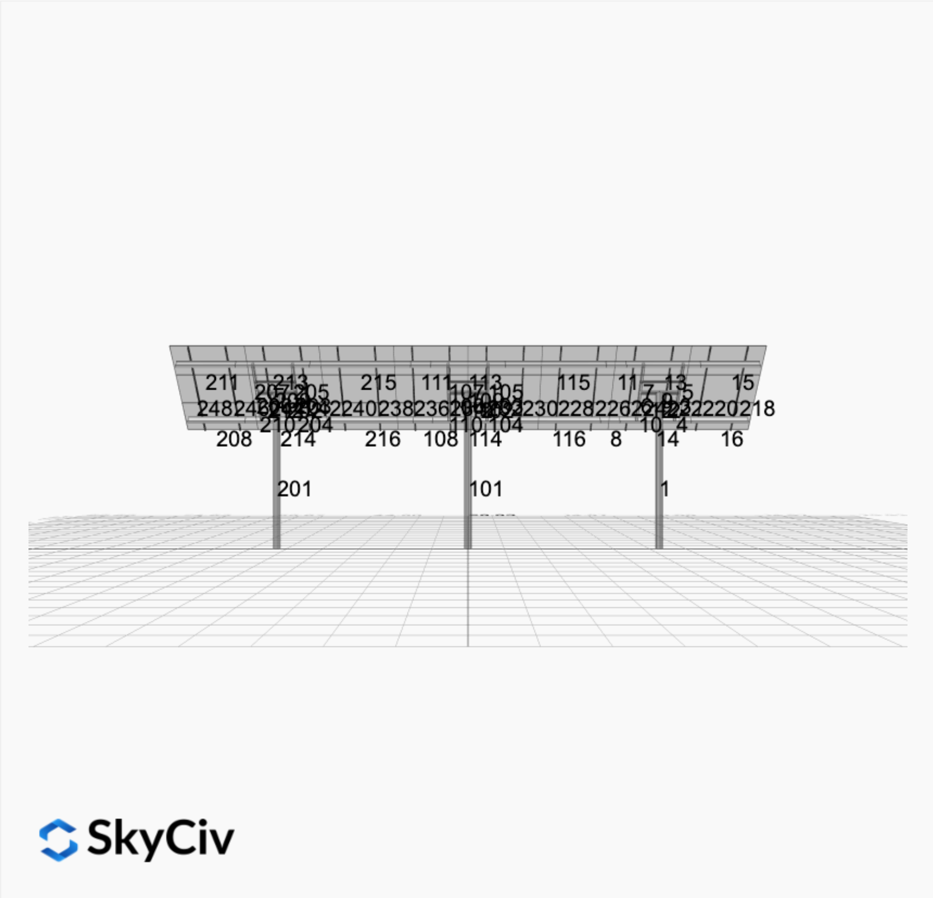


Array Dimensions N/S	11.25 ft
Array Dimensions E/W	59.87 ft
Winter Tilt Angle	47
Front Edge Clearance	12 ft

MT Solar Bill of Materials (3P-19.75-8TOP-HD-72-L-3Hx8W-K0K3)

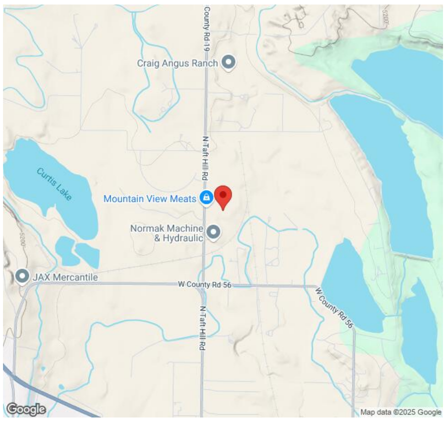
Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-72	72IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	4
MTS-HD-Splice-57	57IN HD Splice	4
MTS-CLAMP-HOOK-4PK	Hook Clamp	8

Rail Bill of Materials

Part	Qty
Rails (134in)	16
Rail Attachment	32

Part	Qty
Module Mid Clamp	32
Module End Clamp	32
Ground Lug	8

Site Details:



Site Address: 5200 N County Rd 19, Fort Collins, CO 80524, USA

Array Specification

Duty Classification:	HD
Module Width:	44.50 in
Module Length:	88.80in
Number of Rows:	3
Number of Columns:	8
Total Number of Modules:	24
Winter Tilt Angle:	47
Front Edge Clearance:	12
Total Array Height at Tilt:	20.23 ft
Total Frame Length:	59.00 ft
Frame Weight:	4430 lbs
Array Dimensions N/S:	11.25 ft
Array Dimensions E/W:	59.87 ft
Rail Length:	135.00 in
Rail Spacing:	3.74 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	16.11 ft
Number of Poles:	3
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.50 ft Pile 2: 8.25 ft Pile 3: 8.50 ft
Foundation Volume:	14.963 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	5200 N County Rd 19, Fort Collins, CO 80524, USA
Wind Speed:	145 mph
Snow Load:	45 psf

Design Disclaimer

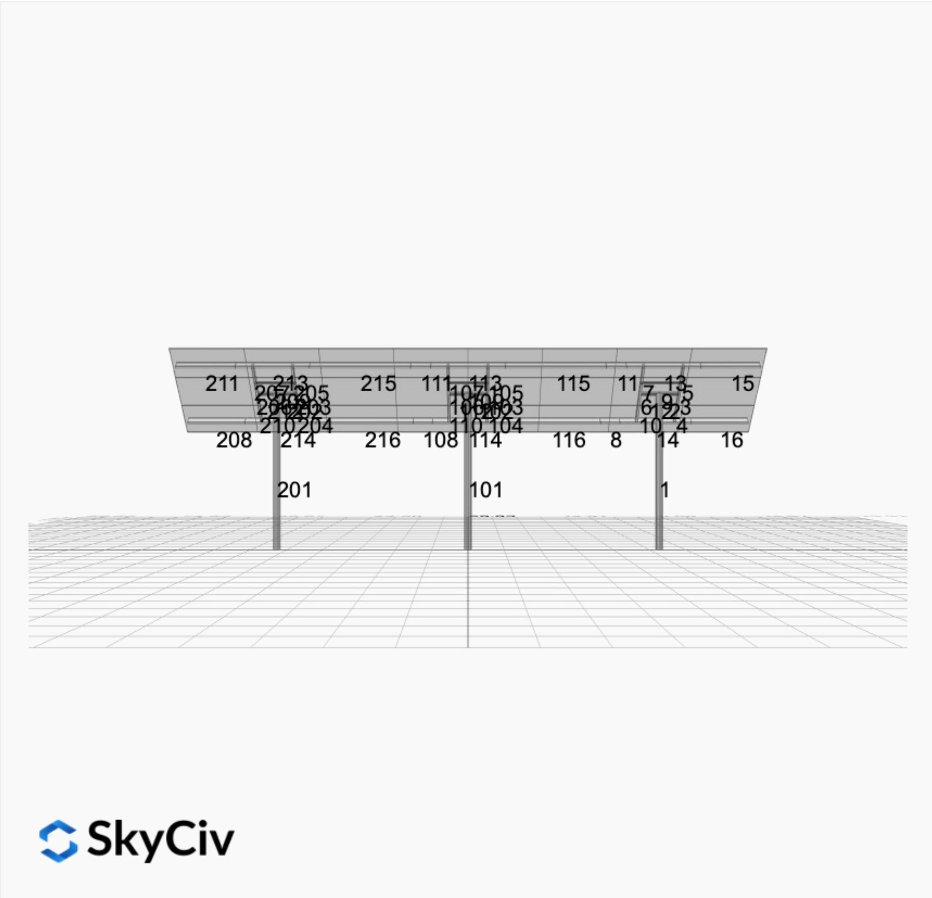
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

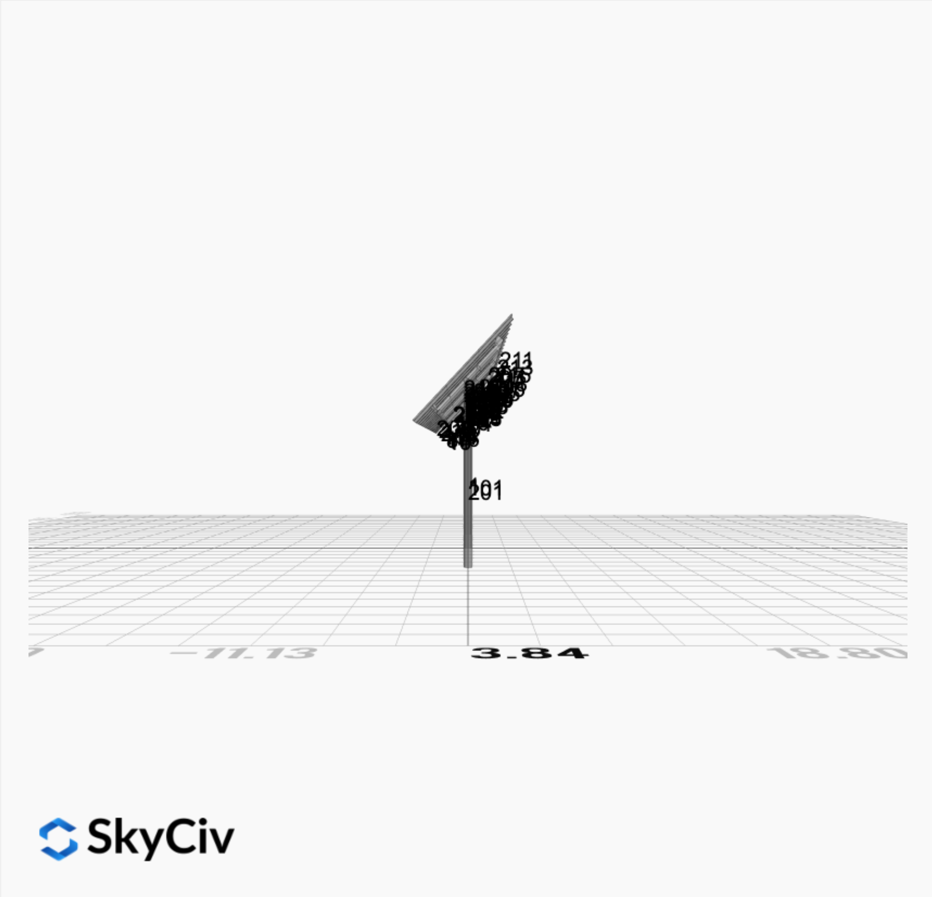
AutoDesigner Input

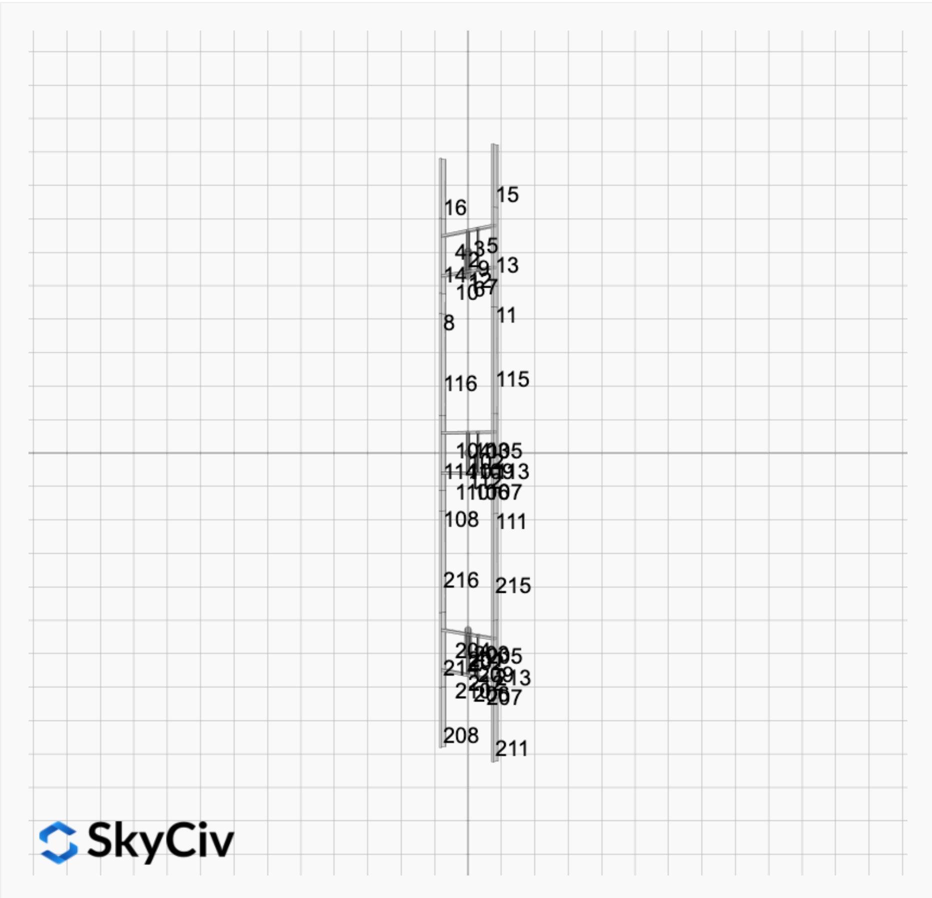
{"wind_speed_override":145,"snow_load_override":45,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"","designer_email":"","designer_phone":"","project_id":"Uthmann Family Feeders - 3x8 - V1Jb","site_address":"5200 N County Rd 19, Fort Collins, CO 80524, USA","module_width":44.5,"module_length":88.8,"number_rows":"3","number_columns":8,"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":47,"ground_clearance":12,"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":true}

Design Notes:

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




211
201
−11.13
3.84
18.80
SkyCiv


16
15
13
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8
116
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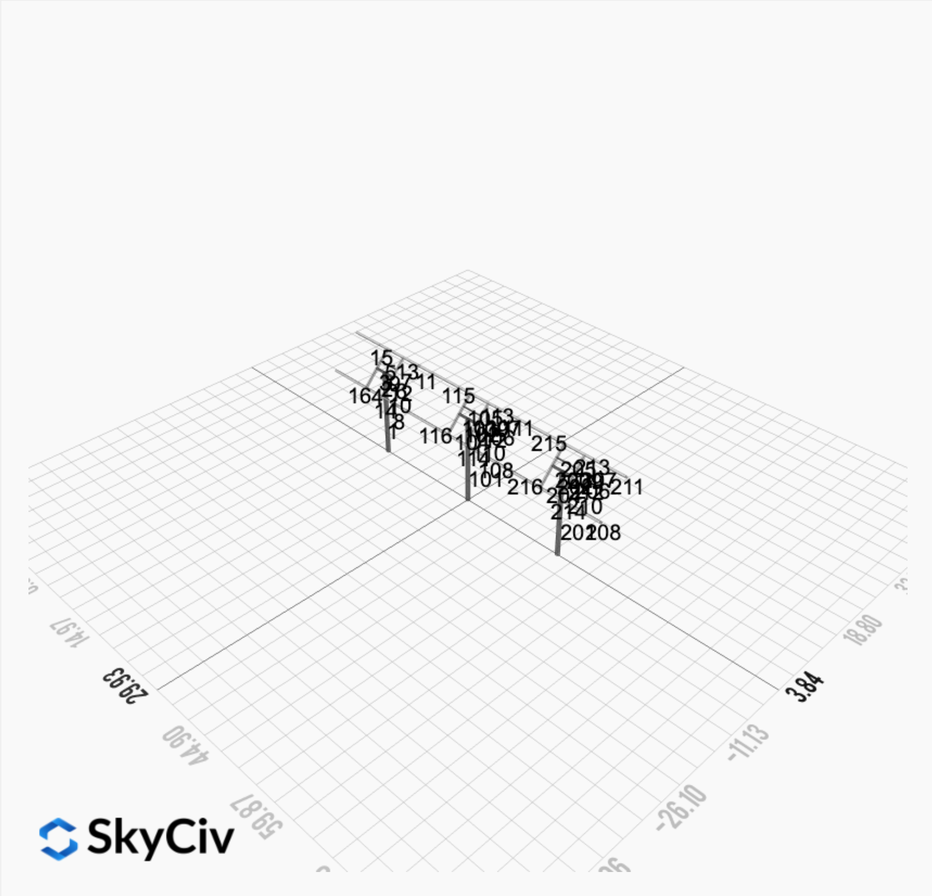
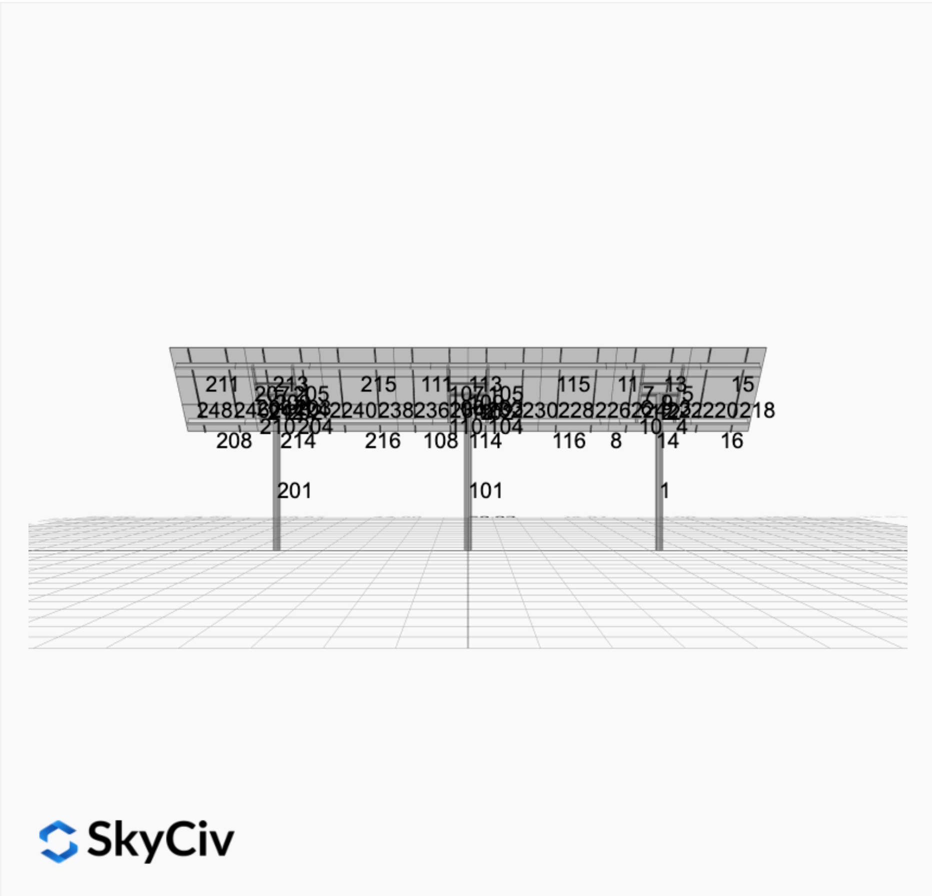
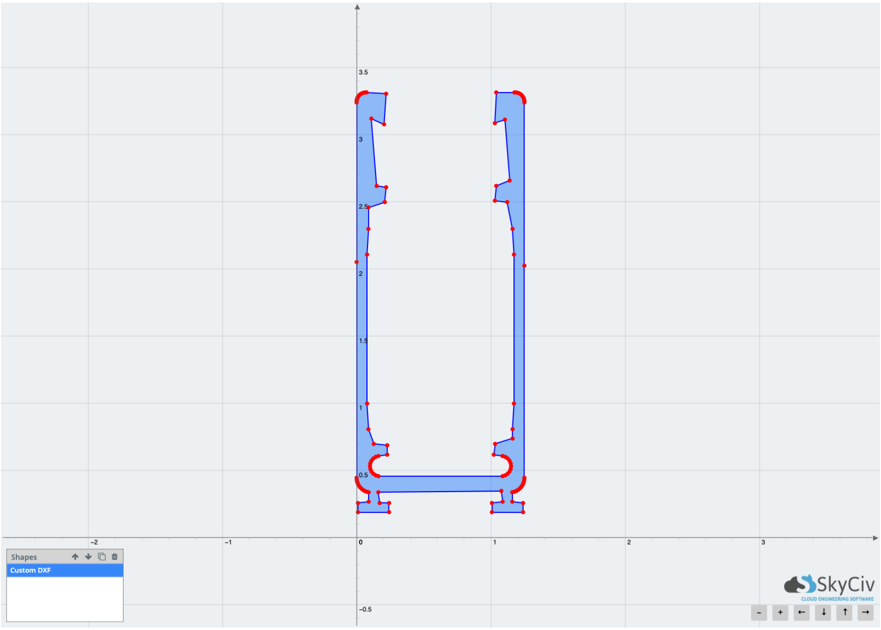
SkyCiv



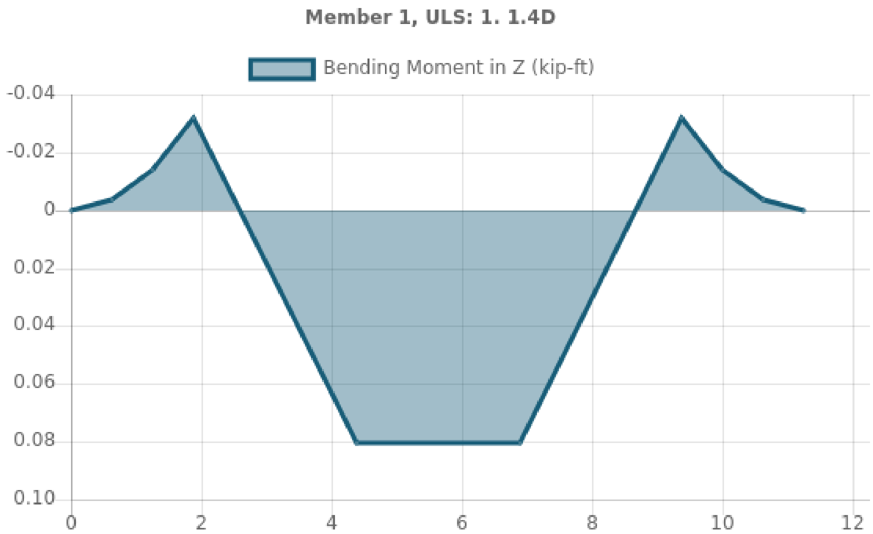


Rail Design Check

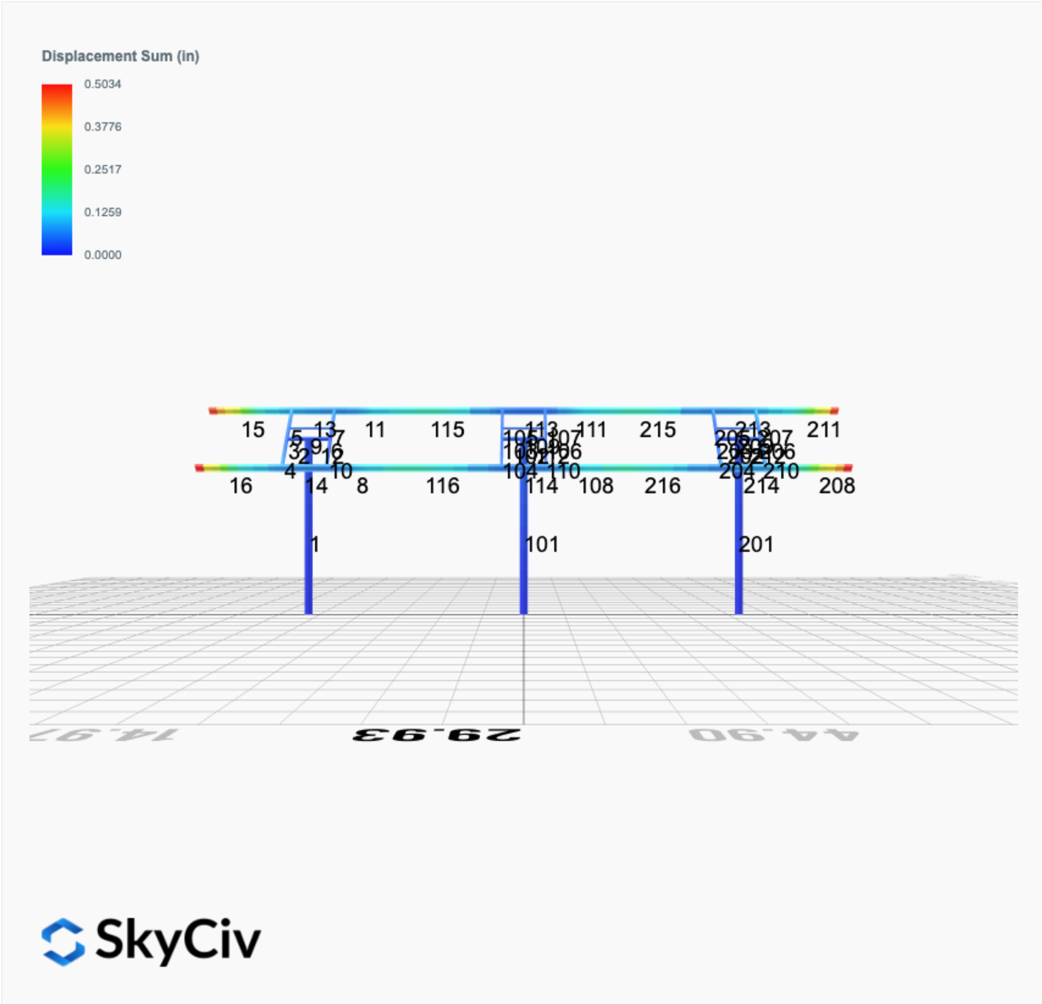
Rail Length: 11.25 ft
Additional Restraints Required: None
Tributary Width: 3.7416666666666667 ft
Material: Aluminium
Density: 169 lb/ft^3
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0290 kip/ft
Snow (Y): -0.0311 kip/ft
Wind uplift Case A: 0.1391 kip/ft
Wind downforce Case A: 0.1391 kip/ft
Dead (Panel load) (X): 0.0120 kip/ft
Dead (Panel load) (Y): -0.0129 kip/ft

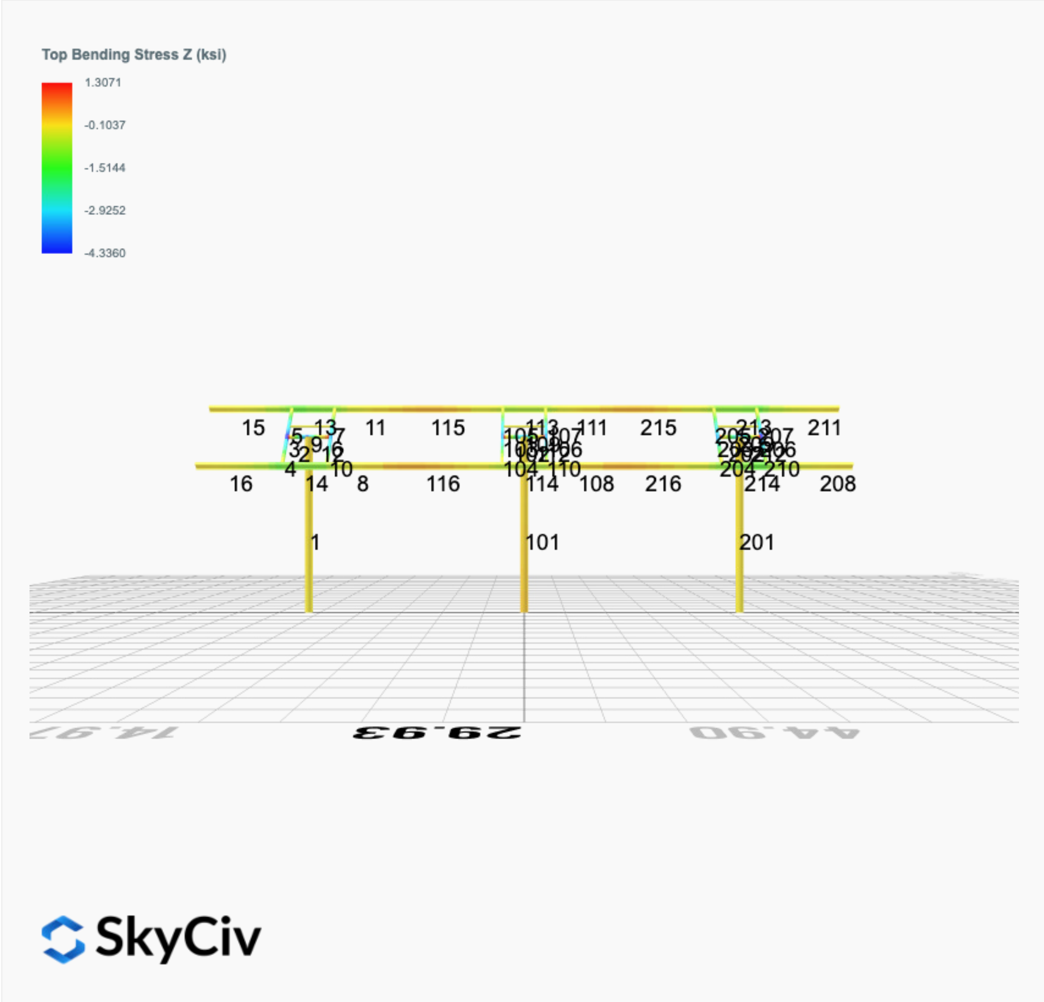


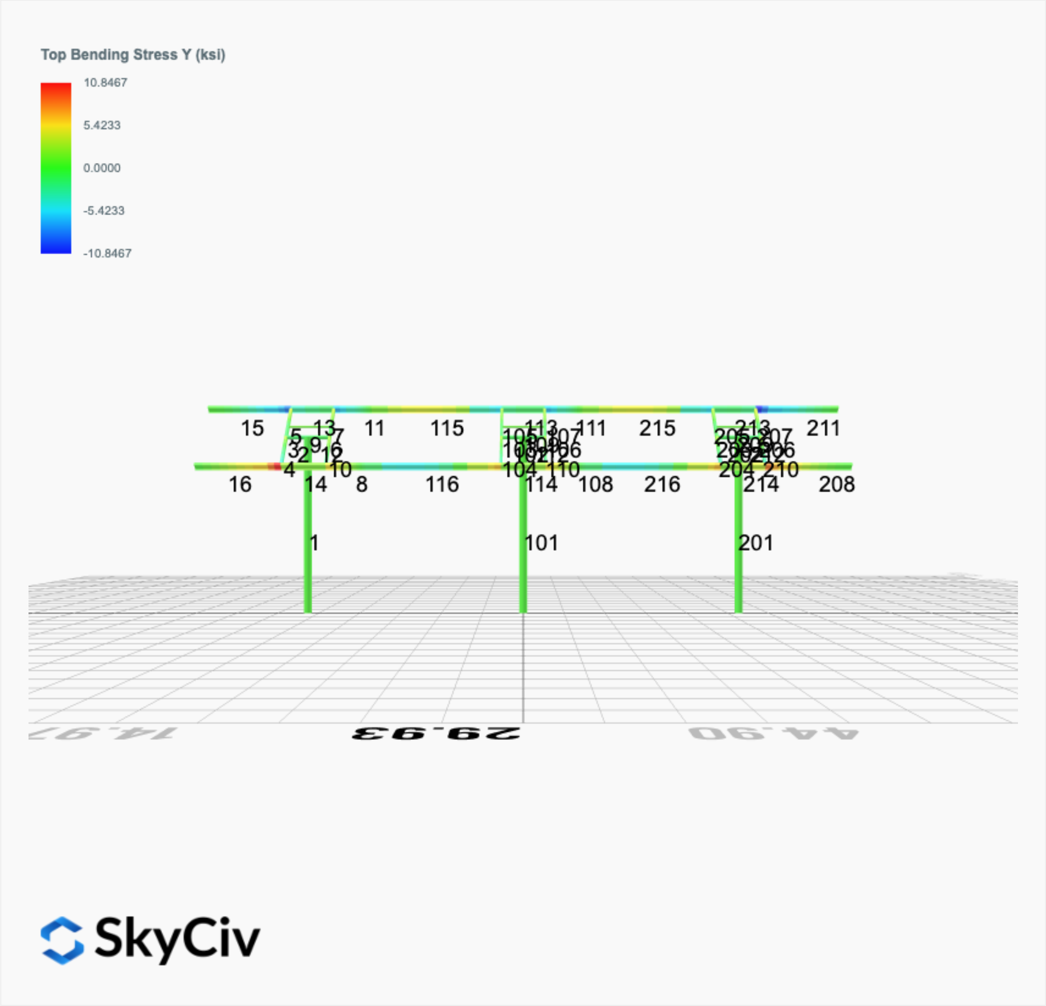
Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	16.33404295	0.473	PASS
Material Yield	34.5	16.33404295	0.473	PASS
Material Strength	37	16.33404295	0.441	PASS

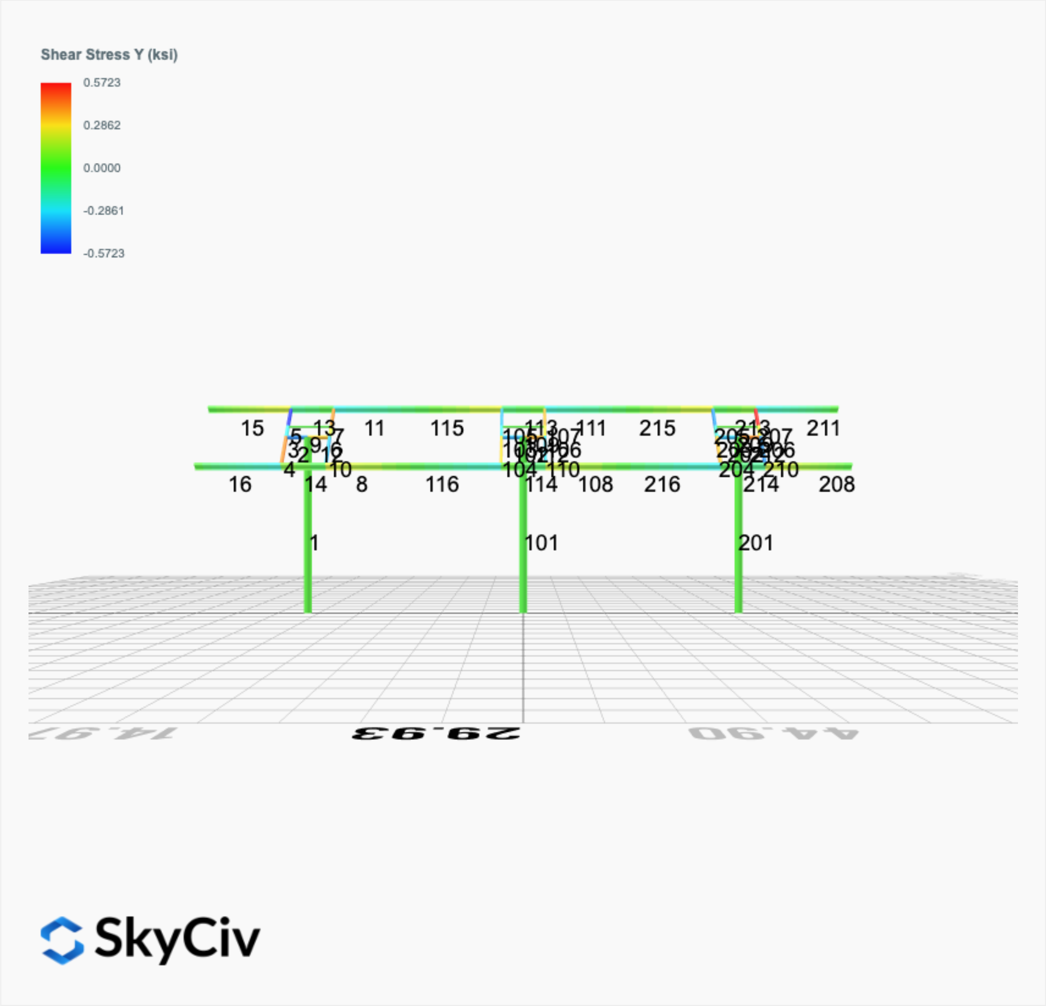


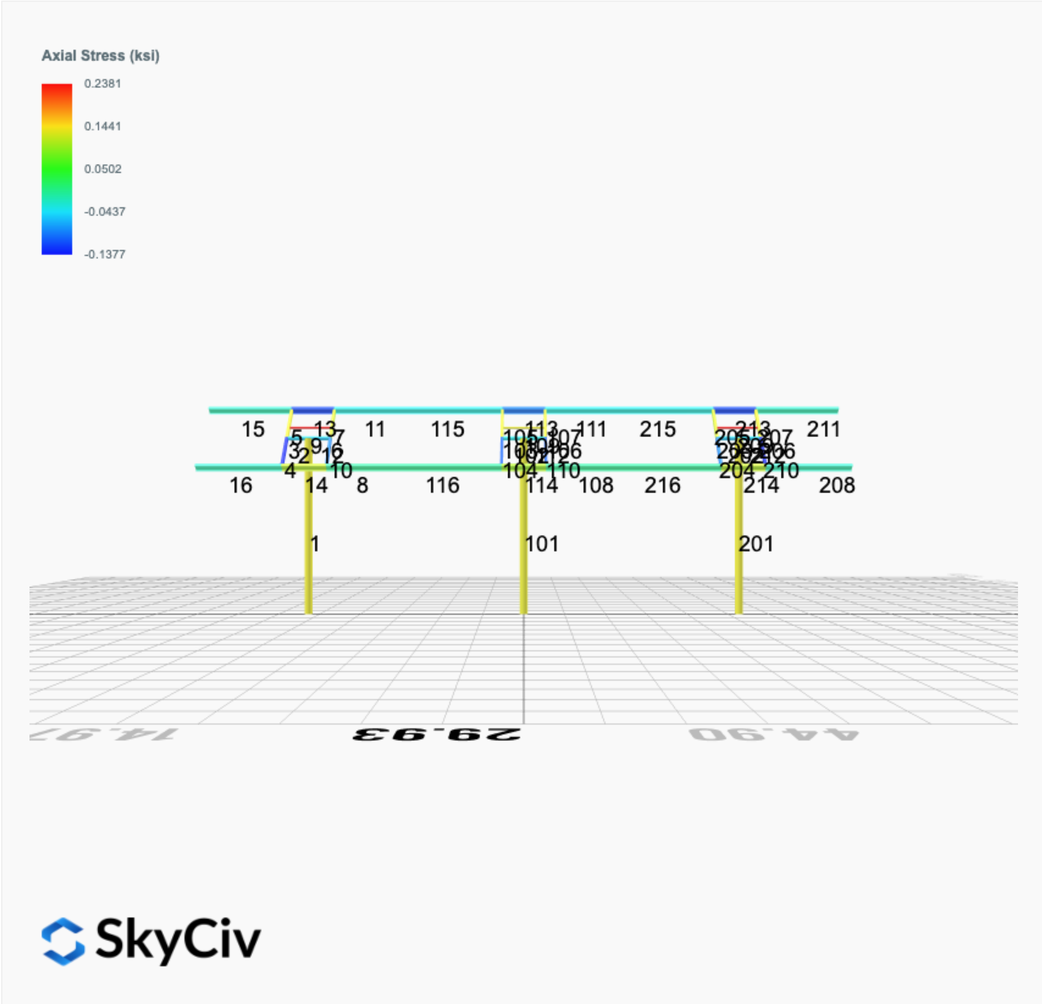
FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
1.3071
-0.1037
-1.5144
-2.9252
-4.3360
15
11
115
111
215
211
16
14
8
116
114
108
216
214
208
1
101
201
SkyCiv


Top Bending Stress Y (ksi)
10.8467
5.4233
0.0000
-5.4233
-10.8467
15
13
11
115
215
211
16
14
8
116
114
108
216
214
208
1
101
201
29.93
44.90
SkyCiv


Shear Stress Y (ksi)
0.5723
0.2862
0.0000
-0.2861
-0.5723
15
13
11
115
111
215
211
16
14
8
116
114
108
216
214
208
1
101
201
29.93
SkyCiv


Axial Stress (ksi)
0.2381
0.1441
0.0502
-0.0437
-0.1377
15
11
115
215
211
16
14
8
116
114
108
216
214
208
1
101
201

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	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 2. D + L	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 3. D + (S or Lr or R)	-0.0295	4.1132	-0.0583	-0.2912	0.2421	0.4546
ULS: 3. D + (S or Lr or R)	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0255	3.6798	-0.0504	-0.2518	0.2094	0.3956
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 5b. D + 0.7E	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0255	3.6798	-0.0504	-0.2518	0.2094	0.3956
ULS: 8. 0.6D + 0.7E	-0.0081	1.4278	-0.0161	-0.0802	0.0667	0.1311
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7399	5.8766	-0.1409	-0.6963	0.7929	61.6149
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7165	-1.1192	0.0852	0.4159	-0.5585	-58.4034
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8202	6.3026	-0.1360	-0.6737	0.7206	46.4429
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0255	3.6798	-0.0504	-0.2518	0.2094	0.3956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7721	1.0557	0.0335	0.1604	-0.2929	-43.5708
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0255	3.6798	-0.0504	-0.2518	0.2094	0.3956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8083	5.0024	-0.1124	-0.5556	0.6225	46.2658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7840	-0.2445	0.0572	0.2785	-0.3911	-43.7479
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0135	2.3796	-0.0268	-0.1337	0.1112	0.2185
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7345	4.9248	-0.1302	-0.6428	0.7484	61.5274
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0081	1.4278	-0.0161	-0.0802	0.0667	0.1311
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7220	-2.0711	0.0959	0.4694	-0.6030	-58.4908
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0081	1.4278	-0.0161	-0.0802	0.0667	0.1311

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	9.5500
Shear X	-6.2332
Shear Z	-0.2389
Moment X	-1.1818
Moment Y (Twist)	1.3383
Moment Z	104.3722

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	6.3026
Shear X	-3.7399
Shear Z	-0.1409
Moment X	-0.6963
Moment Y (Twist)	0.7929
Moment Z	61.6149

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	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 2. D + L	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 3. D + (S or Lr or R)	0.0589	4.0229	-0.0000	0.0000	-0.0000	-0.8337
ULS: 3. D + (S or Lr or R)	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0510	3.6017	-0.0000	0.0000	-0.0000	-0.7186

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 5b. D + 0.7E	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0510	3.6017	-0.0000	0.0000	-0.0000	-0.7186
ULS: 8. 0.6D + 0.7E	0.0163	1.4028	-0.0000	0.0000	0.0000	-0.2239
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.5077	5.5901	-0.0000	0.0000	0.0000	58.1028
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5544	-0.9103	-0.0000	0.0000	0.0000	-56.1906
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6002	6.0407	-0.0000	0.0000	-0.0000	43.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0510	3.6017	-0.0000	0.0000	-0.0000	-0.7186
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6965	1.1654	-0.0000	0.0000	-0.0000	-42.5817
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0510	3.6017	-0.0000	0.0000	-0.0000	-0.7186
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6240	4.7771	-0.0000	0.0000	0.0000	43.4838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6726	-0.0982	-0.0000	0.0000	0.0000	-42.2363
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0271	2.3381	-0.0000	0.0000	0.0000	-0.3732
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.5186	4.6548	-0.0000	0.0000	0.0000	58.2520
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0163	1.4028	-0.0000	0.0000	0.0000	-0.2239
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5436	-1.8455	-0.0000	0.0000	0.0000	-56.0414
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0163	1.4028	-0.0000	0.0000	0.0000	-0.2239

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	9.0695
Shear X	-5.9296
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0002
Moment Z	98.3948

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	6.0407
Shear X	-3.5544
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	58.2520

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 2. D + L	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 3. D + (S or Lr or R)	-0.0295	4.1132	0.0583	0.2912	-0.2421	0.4546
ULS: 3. D + (S or Lr or R)	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0255	3.6798	0.0504	0.2518	-0.2094	0.3956
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 5b. D + 0.7E	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0255	3.6798	0.0504	0.2518	-0.2094	0.3956
ULS: 8. 0.6D + 0.7E	-0.0081	1.4278	0.0161	0.0802	-0.0667	0.1311
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7399	5.8766	0.1409	0.6963	-0.7929	61.6149
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7165	-1.1192	-0.0852	-0.4159	0.5585	-58.4034
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8202	6.3026	0.1360	0.6738	-0.7206	46.4428
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0255	3.6798	0.0504	0.2518	-0.2094	0.3956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7721	1.0557	-0.0335	-0.1604	0.2929	-43.5708
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0255	3.6798	0.0504	0.2518	-0.2094	0.3956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8083	5.0024	0.1124	0.5556	-0.6225	46.2658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7840	-0.2445	-0.0572	-0.2785	0.3911	-43.7479
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0136	2.3796	0.0268	0.1337	-0.1112	0.2185
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7345	4.9248	0.1302	0.6428	-0.7484	61.5274
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0081	1.4278	0.0161	0.0802	-0.0667	0.1311
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7220	-2.0711	-0.0959	-0.4694	0.6030	-58.4908
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0081	1.4278	0.0161	0.0802	-0.0667	0.1311

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	9.5500
Shear X	-6.2332
Shear Z	0.2389
Moment X	1.1818
Moment Y (Twist)	1.3387
Moment Z	104.3732

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	6.3026
Shear X	-3.7399
Shear Z	0.1409
Moment X	0.6963
Moment Y (Twist)	0.7929
Moment Z	61.6149

Project Details

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

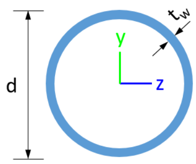
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

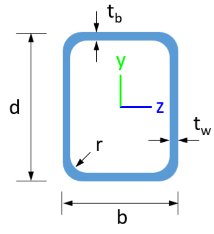
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

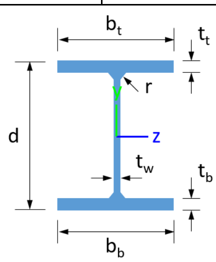
Section Dimensions



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



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



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

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

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
10	8in Pipe Sch 80	12.76	211.43	105.72	105.72	0.00	33.05	33.05
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties

Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	10	33.84	33.84	16.11	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.65,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19	300	200	1
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
8	19	1.33	1.33	2.05	1.88,1.88,1.88,1.88,1.88,1.88,1.86,1.88,1.85,1.88,1.86,1.88,1.86,1.88,1.87,1.88,1.80,1.88,1.86,1.88,1.85,1.88,1.86,1.88,1.86,1.88	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
11	19	1.33	1.33	2.05	1.93,1.92,1.93,1.92,1.93,1.93,1.90,1.92,1.89,1.92,1.90,1.93,1.89,1.93,1.91,1.92,1.83,1.92,1.90,1.93,1.88,1.93,1.90,1.93,1.89,1.93	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10	300	200	1
14	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10	300	200	1
15	19	12.60	12.60	6.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	12.60	12.60	6.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	10	33.84	33.84	16.11	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
104	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
105	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
106	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
107	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
108	19	1.33	1.33	2.05	2.08,2.08,2.08,2.08,2.08,2.08,2.08,2.08,2.07,2.08,2.08,2.08,2.07,2.08,2.08,2.08,2.07,2.08,2.08,2.08,2.07,2.08,2.08,2.08,2.07,2.08	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
111	19	1.33	1.33	2.05	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.35,2.33,2.33,2.33,2.35,2.33,2.33,2.33,2.38,2.33,2.33,2.33,2.35,2.33,2.33,2.33,2.35,2.33	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1

113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.03,1.03,1.04,1.03,1.03,1.03,1.04,1.03,1.04,1.03,1.04,1.03,1.03,1.03,1.04,1.03	300	200	1
114	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
115	19	6.63	6.63	10.20	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
116	19	6.63	6.63	10.20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16	300	200	1
201	10	33.84	33.84	16.11	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.16,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19	300	200	1
204	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.69,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.64,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
205	16	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
206	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.65,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.68	300	200	1
208	19	12.60	12.60	6.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.68,1.65,1.68,1.67,1.69,1.66,1.69,1.67,1.69,1.66,1.69	300	200	1
211	19	12.60	12.60	6.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10	300	200	1
214	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10,1.09,1.10	300	200	1
215	19	6.63	6.63	10.20	1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.28,1.27,1.27,1.27,1.28,1.27,1.27,1.27,1.30,1.27,1.27,1.27,1.28,1.27,1.27,1.27,1.28,1.27	300	200	1
216	19	6.63	6.63	10.20	1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.30,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28,1.28	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	574.32	144.84	123.94	123.94	172.30	172.30
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	123.95	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	123.95	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	85.85	24.92	6.12	40.24	43.62
14	133.20	85.85	24.87	6.12	40.24	43.62
15	133.20	20.65	32.87	6.12	40.24	43.62
16	133.20	20.65	32.87	6.12	40.24	43.62
101	574.32	144.84	123.94	123.94	172.30	172.30

101	574.32	144.84	123.94	123.94	172.30	172.30
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.71	6.12	40.24	43.62
114	133.20	85.85	23.80	6.12	40.24	43.62
115	133.20	69.16	18.17	6.12	40.24	43.62
116	133.20	69.16	17.91	6.12	40.24	43.62
201	574.32	144.84	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	20.65	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	20.65	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	24.91	6.12	40.24	43.62
214	133.20	85.85	24.88	6.12	40.24	43.62
215	133.20	69.16	19.63	6.12	40.24	43.62
216	133.20	69.16	19.76	6.12	40.24	43.62

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.066	0.842	0.021	0.036	0.001	0.885	#13	0.705	Not Required	Pass
2	0.003	0.397	0.308	0.082	0.057	0.706	#13	0.035	Not Required	Pass
3	0.008	0.683	0.046	0.069	0.005	0.713	#13	0.045	Not Required	Pass
4	0.009	0.681	0.168	0.068	0.034	0.718	#13	0.080	Not Required	Pass
5	0.008	0.423	0.173	0.068	0.043	0.448	#13	0.074	Not Required	Pass
6	0.007	0.556	0.039	0.054	0.004	0.573	#13	0.045	Not Required	Pass
7	0.007	0.345	0.135	0.055	0.035	0.370	#13	0.074	Not Required	Pass
8	0.002	0.085	0.115	0.043	0.012	0.175	#21	0.095	Not Required	Pass
9	0.015	0.068	0.073	0.003	0.002	0.145	#13	0.204	Not Required	Pass
10	0.007	0.567	0.140	0.057	0.030	0.643	#13	0.080	Not Required	Pass
11	0.002	0.079	0.115	0.041	0.012	0.172	#21	0.063	Not Required	Pass
12	0.004	0.276	0.247	0.065	0.048	0.525	#13	0.035	Not Required	Pass
13	0.007	0.309	0.410	0.052	0.015	0.630	#21	0.190	Not Required	Pass
14	0.009	0.319	0.410	0.054	0.015	0.630	#21	0.190	Not Required	Pass
15	0.000	0.141	0.246	0.038	0.011	0.346	#21	Not Required	Not Required	Pass

16	0.000	0.141	0.246	0.038	0.011	0.346	#21	Not Required	Not Required	Pass
101	0.063	0.794	0.000	0.034	0.000	0.825	#13	0.705	Not Required	Pass
102	0.002	0.314	0.254	0.069	0.050	0.568	#13	0.035	Not Required	Pass
103	0.007	0.593	0.047	0.059	0.008	0.625	#13	0.045	Not Required	Pass
104	0.007	0.576	0.115	0.058	0.024	0.619	#13	0.080	Not Required	Pass
105	0.007	0.368	0.118	0.059	0.030	0.382	#13	0.074	Not Required	Pass
106	0.007	0.593	0.047	0.059	0.008	0.625	#13	0.045	Not Required	Pass
107	0.007	0.368	0.118	0.059	0.030	0.382	#13	0.074	Not Required	Pass
108	0.002	0.042	0.105	0.036	0.012	0.114	#21	0.095	Not Required	Pass
109	0.009	0.041	0.048	0.001	0.000	0.091	#13	0.204	Not Required	Pass
110	0.007	0.576	0.115	0.058	0.024	0.619	#13	0.080	Not Required	Pass
111	0.002	0.049	0.105	0.037	0.012	0.128	#21	0.063	Not Required	Pass
112	0.002	0.314	0.254	0.069	0.050	0.568	#13	0.035	Not Required	Pass
113	0.007	0.209	0.273	0.048	0.015	0.396	#21	0.190	Not Required	Pass
114	0.009	0.185	0.274	0.047	0.015	0.373	#21	0.286	Not Required	Pass
115	0.002	0.147	0.151	0.037	0.012	0.257	#21	0.316	Not Required	Pass
116	0.004	0.164	0.154	0.036	0.012	0.267	#21	0.473	Not Required	Pass
201	0.066	0.842	0.021	0.036	0.001	0.885	#13	0.705	Not Required	Pass
202	0.004	0.276	0.247	0.065	0.048	0.525	#13	0.035	Not Required	Pass
203	0.007	0.556	0.039	0.054	0.004	0.573	#13	0.045	Not Required	Pass
204	0.007	0.567	0.140	0.057	0.030	0.643	#13	0.080	Not Required	Pass
205	0.007	0.345	0.135	0.055	0.035	0.370	#13	0.074	Not Required	Pass
206	0.008	0.683	0.046	0.069	0.005	0.713	#13	0.045	Not Required	Pass
207	0.008	0.423	0.173	0.068	0.043	0.448	#13	0.074	Not Required	Pass
208	0.000	0.141	0.246	0.038	0.011	0.346	#21	Not Required	Not Required	Pass
209	0.015	0.068	0.073	0.003	0.002	0.145	#13	0.204	Not Required	Pass
210	0.009	0.681	0.168	0.068	0.034	0.718	#13	0.080	Not Required	Pass
211	0.000	0.141	0.246	0.038	0.011	0.346	#21	Not Required	Not Required	Pass
212	0.003	0.397	0.308	0.082	0.057	0.706	#13	0.035	Not Required	Pass
213	0.007	0.309	0.410	0.052	0.015	0.630	#21	0.190	Not Required	Pass
214	0.009	0.319	0.410	0.054	0.015	0.630	#21	0.286	Not Required	Pass
215	0.002	0.139	0.152	0.041	0.012	0.252	#21	0.316	Not Required	Pass
216	0.004	0.149	0.153	0.043	0.012	0.260	#21	0.473	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comresssion)

M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

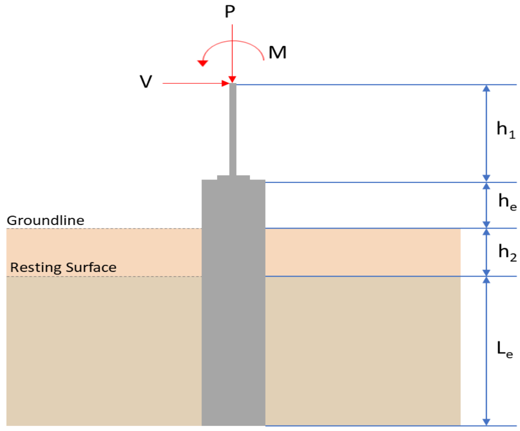
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.041	9.069
V_x (kip)	-3.554	-5.930
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	58.252	98.395

Material Properties

$f'_{ck} = 2.5$ 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,

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

$$H_o = \frac{(-3.554\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.56592\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(58.252 \text{ kipft}) + ((-3.554 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.2758 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.8123 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.8123 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.812 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.812 \text{ ft})}{(8.25 \text{ ft})}$$

$$Ratio = 0.94691$$

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

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

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

$$q = \frac{(6.041 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37756 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.37756 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.18878$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 2.0625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.56592\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 9.2758\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.2758\ \mathrm{kipft/ft}) \times (8.25\ \mathrm{ft})) + \left(3 \times (-0.56592\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft})^2\right)}{(6 \times (9.2758\ \mathrm{kipft/ft})) + (4 \times (-0.56592\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))}$$

$$a = 5.6727\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times \left[(4 \times (9.2758\ \mathrm{kipft/ft})) + (3 \times (-0.56592\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))\right]^2}{(8.25\ \mathrm{ft})^2 \times \left[(3 \times (9.2758\ \mathrm{kipft/ft})) + (2 \times (-0.56592\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))\right]}$$

$$p = 0.31792\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times \left[(2 \times (9.2758\ \mathrm{kipft/ft})) + ((-0.56592\ \mathrm{kip/ft}) \times (8.25\ \mathrm{ft}))\right]}{(8.25\ \mathrm{ft})^2}$$

$$s = 1.2238\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.6727\ \mathrm{ft})}{2}$$

$$p_a = 0.42546\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.31792\ \mathrm{kip/ft^2}\right)}{\left(0.42546\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.74725$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (8.25\ \mathrm{ft})$$

$$p_s = 1.2375\ \mathrm{kip/ft^2}$$

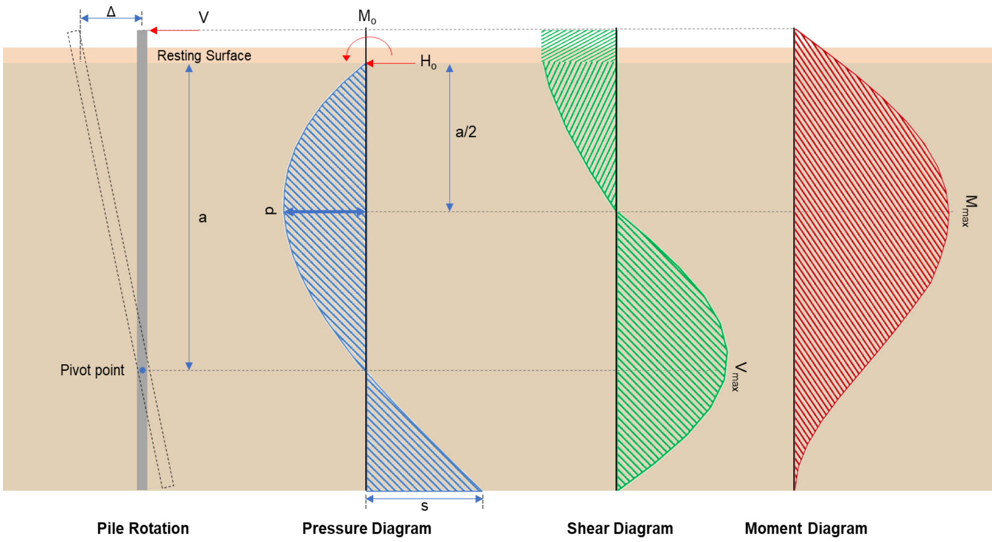
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.2238\ \mathrm{kip/ft^2}\right)}{\left(1.2375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.98895$$

Status: **PASS**
Ratio: **0.990**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.93\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.94427\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(98.395\ \text{kipft}) + ((-5.93\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 15.668\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.668\ \text{kipft/ft})}{(-0.94427\ \text{kip/ft})}$$

$$E = 16.593\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.668\ \text{kipft/ft}) \times (8.25\ \text{ft})) + \left(3 \times (-0.94427\ \text{kip/ft}) \times (8.25\ \text{ft})^2\right)}{(6 \times (15.668\ \text{kipft/ft})) + (4 \times (-0.94427\ \text{kip/ft}) \times (8.25\ \text{ft}))}$$

$$a = 5.6712\ \text{ft}$$

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

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

$$V_{max} = ((-0.94427\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.593\ \text{ft})}{(8.25\ \text{ft})} + 3\right) \times \left(\frac{(5.6712\ \text{ft})}{(8.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.593\ \text{ft})}{(8.25\ \text{ft})} + 2\right) \times \left(\frac{(5.6712\ \text{ft})}{(8.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} \quad 15.036\ \text{kip}$$

$V_{max} = 15.930$ kip

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

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

$$M_{max} = ((-0.94427\ \text{kip/ft}) \times (48\ \text{in}) \times (8.25\ \text{ft})) \times \left[\left(\frac{(16.593\ \text{ft})}{(8.25\ \text{ft})} + \frac{(5.6712\ \text{ft})}{2 \times (8.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (16.593\ \text{ft})}{(8.25\ \text{ft})} + 3\right) \times \left(\frac{(5.6712\ \text{ft})}{2 \times (8.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.593\ \text{ft})}{(8.25\ \text{ft})} + 2\right) \times \left(\frac{(5.6712\ \text{ft})}{2 \times (8.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 62.901\ \text{kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5$ ksi - Concrete strength,
$f_{yk} = 60$ 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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.069\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.295\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.295\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.069\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.00339$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.069\ \text{kip} \rightarrow 9069\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9069 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.69 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.69 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.69 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.69 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.88 \text{ kip}$$

Considering x-direction:

$V_{max} = 15.936 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(15.936 \text{ kip})}{(110.88 \text{ kip})}$$

$$Ratio = 0.14372$$

Status: **PASS**
Ratio: **0.140**

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:
$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 62.901\text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(62.901\text{ kipft})}{(249.6\text{ kipft})}$$

$$Ratio = 0.25201$$

Status: **PASS**
Ratio: **0.250**

REFERENCES	CALCULATIONS	RESULTS

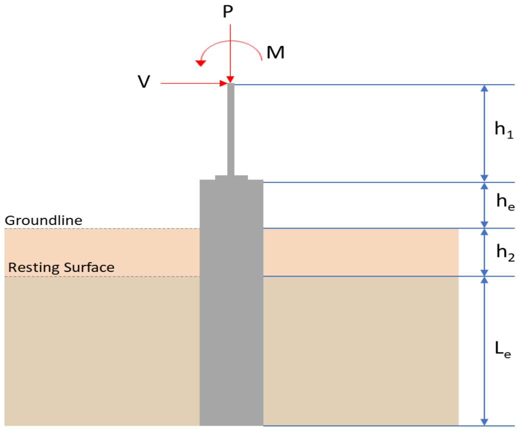
SkyCiv Foundation Design

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
$L = 8.5\ \text{ft}$ - 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)	6.303	9.550
V_x (kip)	-3.740	-6.233
V_z (kip)	-0.141	-0.239
M_x (kipft)	-0.696	-1.182
M_z (kipft)	61.615	104.372

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,

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

$$H_o = \frac{(-3.74\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.59554\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.615\ \text{kipft}) + ((-3.74\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 9.8113\ \text{kipft/ft}$$

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:
$L_{e,x} = 7.9428\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.141\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.022452\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.696\ \text{kipft}) + ((-0.141\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.11083\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.8537\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.9428\ \text{ft}), (1.8537\ \text{ft})]$$

$$L_{e,req} = 7.943\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 8.5\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.943\ \text{ft})}{(8.5\ \text{ft})}$$

$$Ratio = 0.93447$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(6.303 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39394 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.39394 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.19697$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.59554 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 9.8113 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.8113 \text{ kipft/ft}) \times (8.5 \text{ ft})) + \left(3 \times (-0.59554 \text{ kip/ft}) \times (8.5 \text{ ft})^2\right)}{(6 \times (9.8113 \text{ kipft/ft})) + (4 \times (-0.59554 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.848 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.8113 \text{ kipft/ft})) + (3 \times (-0.59554 \text{ kip/ft}) \times (8.5 \text{ ft}))]^{\,2}}{(8.5 \text{ ft})^2 \times [(3 \times (9.8113 \text{ kipft/ft})) + (2 \times (-0.59554 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.31117 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (9.8113 \text{ kipft/ft})) + ((-0.59554 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2092 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.848 \text{ ft})}{2}$$

$$p_a = 0.4386 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.31117\ \text{kip/ft}^2\right)}{\left(0.4386\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.70947$$

Status: **PASS**
Ratio: **0.710**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.5\ \text{ft})$$

$$p_s = 1.275\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.2092\ \text{kip/ft}^2\right)}{\left(1.275\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.94837$$

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

Considering z-direction:

$H_o = -0.022452\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.11083\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11083\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.022452\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{(6 \times (0.11083\ \text{kipft/ft})) + (4 \times (-0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 6.0452\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11083\ \text{kipft/ft})) + (3 \times (-0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]^2}{(8.5\ \text{ft})^2 \times [(3 \times (0.11083\ \text{kipft/ft})) + (2 \times (-0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]}$$

$$p = -0.0035228\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.11083\ \text{kipft/ft})) + ((-0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]}{(8.5\ \text{ft})^2}$$

$$s = 0.0025588\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(6.0452\ \text{ft})}{2}$$

$$p_a = 0.45339\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.0035228\ \text{kip/ft}^2\right)}{\left(0.45339\ \text{kip/ft}^2\right)}$$

$$Ratio = -0.0077698$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.5\ \text{ft})$$

$$p_s = 1.275\ \text{kip/ft}^2$$

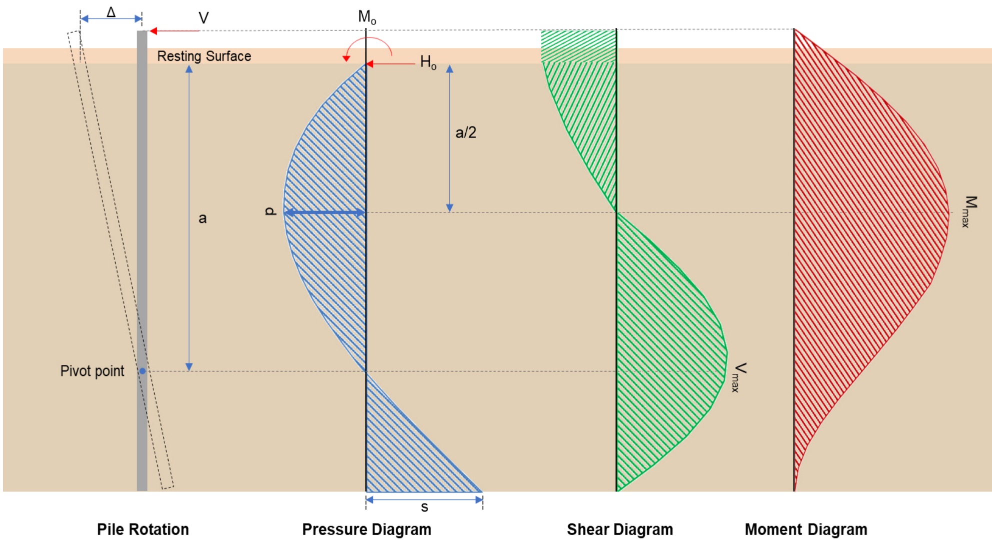
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.0025588\ \text{kip/ft}^2\right)}{\left(1.275\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.0020069$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.233\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.99252\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(104.37\ \text{kipft}) + ((-6.233\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 16.62\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.62\ \text{kipft/ft})}{(-0.99252\ \text{kip/ft})}$$

$$E = 16.745\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.62\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.99252\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{\ldots}$$

$$a = \frac{}{(6 \times (16.62\ \text{kipft/ft})) + (4 \times (-0.99252\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 5.8458\ \text{ft}$$

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

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

$$V_{max} = ((-0.99252\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.745\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(5.8458\ \text{ft})}{(8.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.745\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(5.8458\ \text{ft})}{(8.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 16.46\ \text{kip}$$

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

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

$$M_{max} = ((-0.99252\ \text{kip/ft}) \times (48\ \text{in}) \times (8.5\ \text{ft})) \times \left[\left(\frac{(16.745\ \text{ft})}{(8.5\ \text{ft})} + \frac{(5.8458\ \text{ft})}{2 \times (8.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (16.745\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(5.8458\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.745\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(5.8458\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 66.886\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.239\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.038057\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.182\ \text{kipft}) + ((-0.239\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.18822\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.18822\ \text{kipft/ft})}{(-0.038057\ \text{kip/ft})}$$

$$E = 4.9456\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18822\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.038057\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{(6 \times (0.18822\ \text{kipft/ft})) + (4 \times (-0.038057\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 6.0449\ \text{ft}$$

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

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

$$V_{max} = ((-0.038057\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(6.0449\ \text{ft})}{(8.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(6.0449\ \text{ft})}{(8.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.25793\ \text{kip}$$

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

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

$$M_{max} = ((-0.038057\ \text{kip/ft}) \times (48\ \text{in}) \times (8.5\ \text{ft})) \times \left[\left(\frac{(4.9456\ \text{ft})}{(8.5\ \text{ft})} + \frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.98053\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.55\ \text{kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.279\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.279\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.55\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0035698$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\ \text{in}$ - Effective width,

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.55 \text{ kip} \rightarrow 9550 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9550 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.76 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.76 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.76 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.76 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.92 \text{ kip}$$

Considering x-direction:

$V_{max} = 16.46 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(16.46\ \text{kip})}{(110.92\ \text{kip})}$$

$$Ratio = 0.14839$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.25793\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.25793\ \text{kip})}{(110.92\ \text{kip})}$$

$$Ratio = 0.0023253$$

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:
$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 66.886\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(66.886\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.26797$$

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

Considering z-direction:

$M_{max} = 0.98053\ \text{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(0.98053 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0039284$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

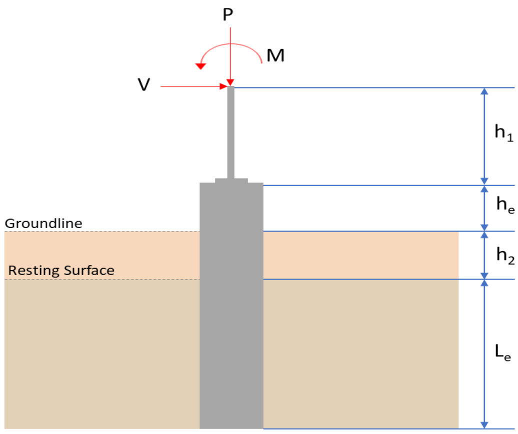
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 8.5\ \text{ft}$ - 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)	6.303	9.550
V_x (kip)	-3.740	-6.233
V_z (kip)	0.141	0.239
M_x (kipft)	0.696	1.182
M_z (kipft)	61.615	104.373

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,

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

$$H_o = \frac{(-3.74\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.59554\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(61.615 \text{ kipft}) + ((-3.74 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.8113 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.9428 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.141 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.022452 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.696 \text{ kipft}) + ((0.141 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11083 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.286 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.9428 \text{ ft}), (2.286 \text{ ft})]$$

$$L_{e,req} = 7.943 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.943 \text{ ft})}{(8.5 \text{ ft})}$$

$$Ratio = 0.93447$$

Status: **PASS**
Ratio: **0.930**

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

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

$$q = \frac{(6.303\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.39394\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.39394\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.19697$$

Status: **PASS**
Ratio: **0.200**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 2.125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.59554\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 9.8113\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.8113\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.59554\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{(6 \times (9.8113\ \text{kipft/ft})) + (4 \times (-0.59554\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 5.848\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.8113\ \text{kipft/ft})) + (3 \times (-0.59554\ \text{kip/ft}) \times (8.5\ \text{ft}))]^{\ 2}}{(8.5\ \text{ft})^2 \times [(3 \times (9.8113\ \text{kipft/ft})) + (2 \times (-0.59554\ \text{kip/ft}) \times (8.5\ \text{ft}))]}$$

$$p = 0.31117\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (9.8113\ \text{kipft/ft})) + ((-0.59554\ \text{kip/ft}) \times (8.5\ \text{ft}))]}{(8.5\ \text{ft})^2}$$

$$s = 1.2092\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(5.848\ \text{ft})}{2}$$

$$p_a = 0.4386\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.31117\ \text{kip/ft}^2\right)}{\left(0.4386\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.70947$$

Status: **PASS**
Ratio: **0.710**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.5\ \text{ft})$$

$$p_s = 1.275\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.2092\ \text{kip/ft}^2\right)}{\left(1.275\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.94837$$

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

Considering z-direction:

$H_o = 0.022452\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.11083\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11083\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (0.022452\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{(6 \times (0.11083\ \text{kipft/ft})) + (4 \times (0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))}$$

$$a = 6.0452\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11083\ \text{kipft/ft})) + (3 \times (0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]^2}{(8.5\ \text{ft})^2 \times [(3 \times (0.11083\ \text{kipft/ft})) + (2 \times (0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]}$$

$$p = 0.015\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.11083\ \text{kipft/ft})) + ((0.022452\ \text{kip/ft}) \times (8.5\ \text{ft}))]}{(8.5\ \text{ft})^2}$$

$$s = 0.034256\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(6.0452\ \text{ft})}{2}$$

$$p_a = 0.45339\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.015\ \text{kip/ft}^2\right)}{\left(0.45339\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.033083$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \text{psf/ft}) \times (8.5\ \text{ft})$$

$$p_s = 1.275\ \text{kip/ft}^2$$

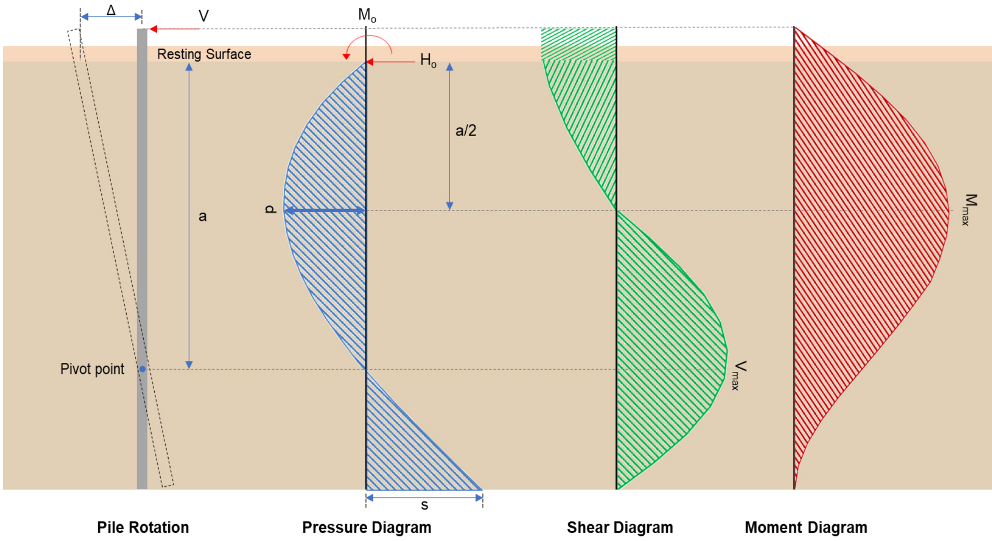
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.034256\ \text{kip/ft}^2)}{(1.275\ \text{kip/ft}^2)}$$

$$Ratio = 0.026867$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.233\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.99252\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(104.37\ \text{kipft}) + ((-6.233\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 16.62\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(16.62\ \text{kipft/ft})}{(-0.99252\ \text{kip/ft})}$$

$$E = 16.745\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (16.62\ \text{kipft/ft}) \times (8.5\ \text{ft})) + \left(3 \times (-0.99252\ \text{kip/ft}) \times (8.5\ \text{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (16.62\ \mathrm{kipft/ft})) + (4 \times (-0.99252\ \mathrm{kip/ft}) \times (8.5\ \mathrm{ft}))}$$

$$a = 5.8458\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.99252\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (16.745\ \mathrm{ft})}{(8.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.8458\ \mathrm{ft})}{(8.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (16.745\ \mathrm{ft})}{(8.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.8458\ \mathrm{ft})}{(8.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 16.46\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.99252\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (8.5\ \mathrm{ft})) \times \left[\left(\frac{(16.745\ \mathrm{ft})}{(8.5\ \mathrm{ft})} + \frac{(5.8458\ \mathrm{ft})}{2 \times (8.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (16.745\ \mathrm{ft})}{(8.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.8458\ \mathrm{ft})}{2 \times (8.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (16.745\ \mathrm{ft})}{(8.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.8458\ \mathrm{ft})}{2 \times (8.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 66.887\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.239\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = 0.038057\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.182\ \mathrm{kipft}) + ((0.239\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.18822\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.18822\ \mathrm{kipft/ft})}{(0.038057\ \mathrm{kip/ft})}$$

$$E = 4.9456\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.18822\ \mathrm{kipft/ft}) \times (8.5\ \mathrm{ft})) + \left(3 \times (0.038057\ \mathrm{kip/ft}) \times (8.5\ \mathrm{ft})^2\right)}{(6 \times (0.18822\ \mathrm{kipft/ft})) + (4 \times (0.038057\ \mathrm{kip/ft}) \times (8.5\ \mathrm{ft}))}$$

$$a = 6.0449\ \mathrm{ft}$$

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

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

$$V_{max} = ((0.038057\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(6.0449\ \text{ft})}{(8.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(6.0449\ \text{ft})}{(8.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.25793\ \text{kip}$$

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

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

$$M_{max} = ((0.038057\ \text{kip/ft}) \times (48\ \text{in}) \times (8.5\ \text{ft})) \times \left[\left(\frac{(4.9456\ \text{ft})}{(8.5\ \text{ft})} + \frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 3\right) \times \left(\frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.9456\ \text{ft})}{(8.5\ \text{ft})} + 2\right) \times \left(\frac{(6.0449\ \text{ft})}{2 \times (8.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.98053\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.55\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.279\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.279\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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,

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

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

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

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,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.55\text{ kip})}{(2675.2\text{ kip})}$$

$$Ratio = 0.0035698$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

$b_w = 48\text{ in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$,
22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.55 \text{ kip} \rightarrow 9550 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9550 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.76 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.76 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.76 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.76 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.92 \text{ kip}$$

Considering x-direction:

$V_{max} = 16.46 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(16.46 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.14839$$

Status: **PASS**
Ratio: **0.150**

Considering z-direction:

$V_{max} = 0.25793 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.25793 \text{ kip})}{(110.92 \text{ kip})}$$

$$Ratio = 0.0023253$$

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:
$\phi M_{n,1}$

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

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

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

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:

$M_{max} = 66.887 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(66.887 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.26798$$

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

Considering z-direction:

$M_{max} = 0.98053 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(0.98053 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0039284$$	Status: **PASS** Ratio: **0.000**