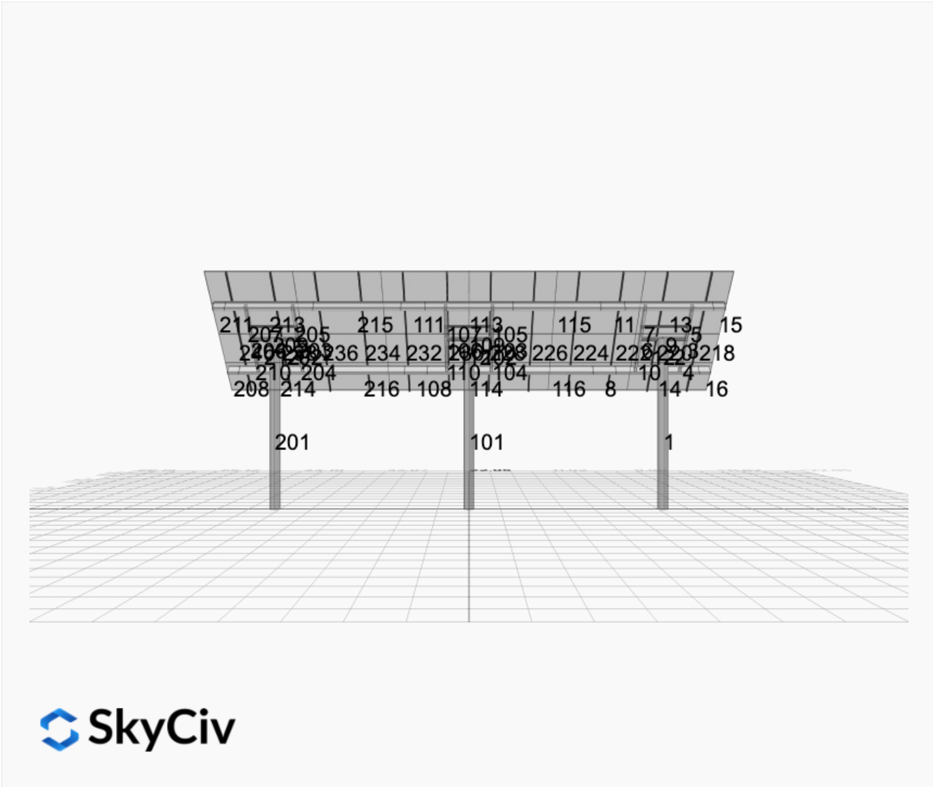
Project Details

Project Name: W-12514-Guffey 10k - V1Jb
Location: PF77+G3, Guffey, CO 80820, USA
Unique ID: 3P-17-10TOP-XD-12-L-4Hx6W-AD77
Dealer: ______________________________

Date: Wed Sep 03 2025
Number of Modules: 24
Number of Poles: 3
Date Sold: ______________________________



Array Dimensions N/S	13.87 ft
Array Dimensions E/W	44.10 ft
Winter Tilt Angle	45
Front Edge Clearance	10 ft

MT Solar Bill of Materials (3P-17-10TOP-XD-12-L-4Hx6W-AD77)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	3
MTS-HF-XD	H-Frame Assembly-XD	3
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-57	57IN XD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	6

Rail Bill of Materials

Part	Qty
Rails (166in)	12
Rail Attachment	24
Module Mid Clamp	36
Module End Clamp	24
Ground Lug	6

Site Details:

Site Address: PF77+G3, Guffey, CO 80820, USA

Array Specification

Duty Classification:	XD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	4
Number of Columns:	6
Total Number of Modules:	24
Winter Tilt Angle:	45
Front Edge Clearance:	10
Total Array Height at Tilt:	19.81 ft
Total Frame Length:	43.50 ft
Module Info/Notes:	
Array Dimensions N/S:	13.87 ft
Array Dimensions E/W:	44.10 ft
Rail Length:	166.40 in
Rail Spacing:	3.68 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	14.90 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 7.75 ft Pile 3: 7.50 ft
Foundation Volume:	13.481 y^3

Site Info

Risk Category:	II
Exposure:	C
Soil Classification:	sand
Site Location:	PF77+G3, Guffey, CO 80820, USA
Wind Speed:	110 mph
Snow Load:	68 psf

Design Disclaimer

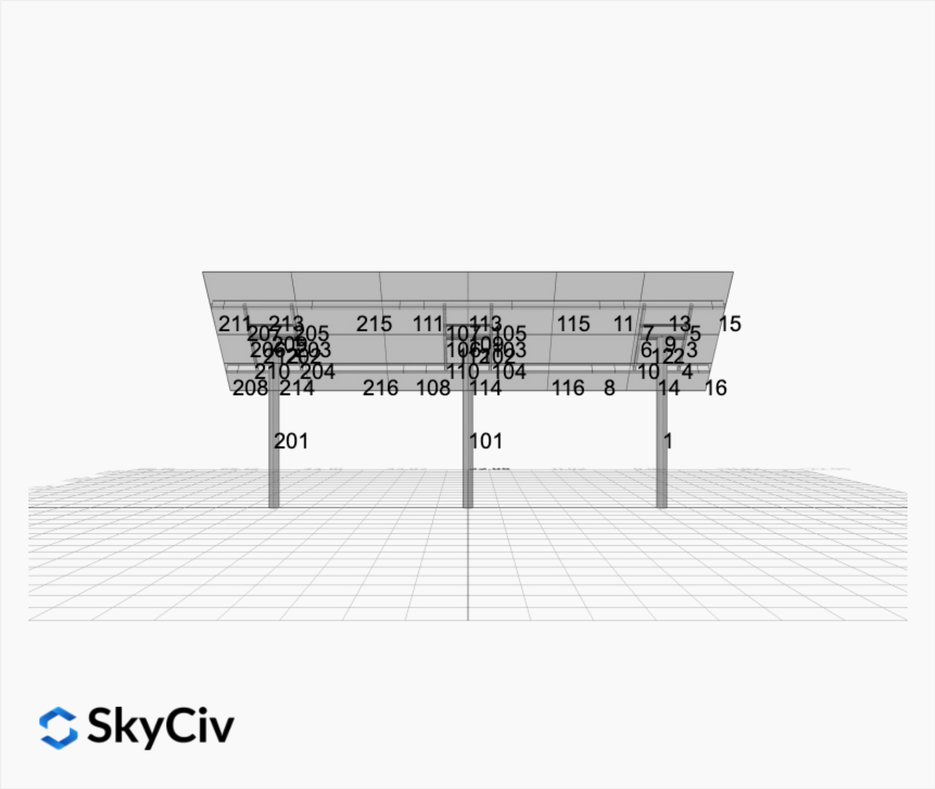
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

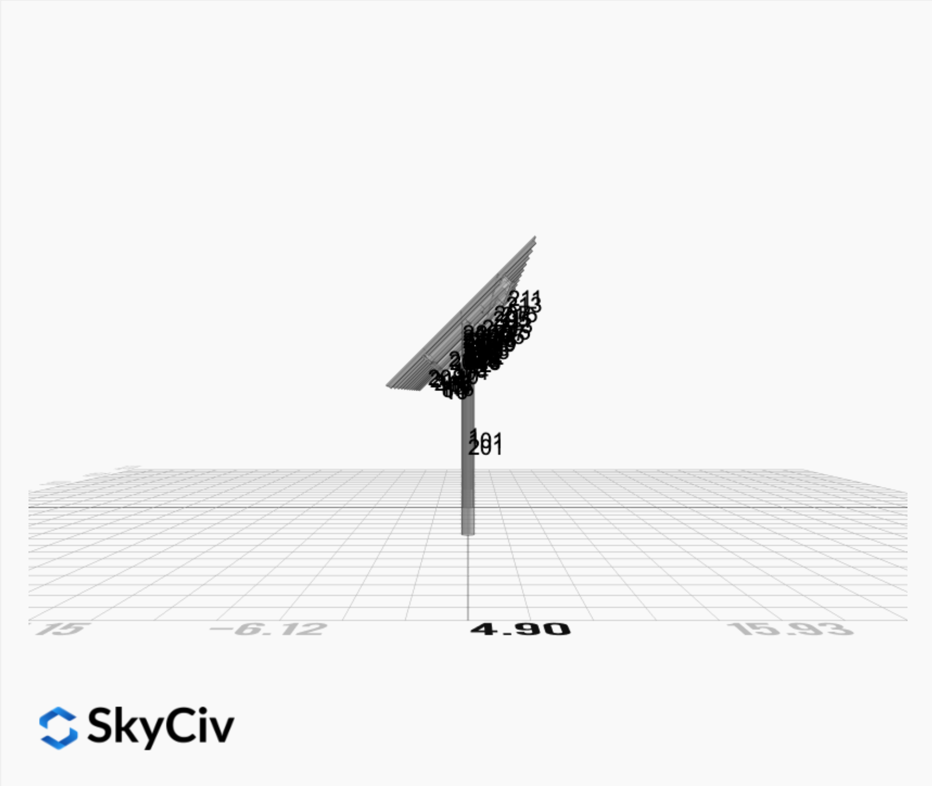
AutoDesigner Input

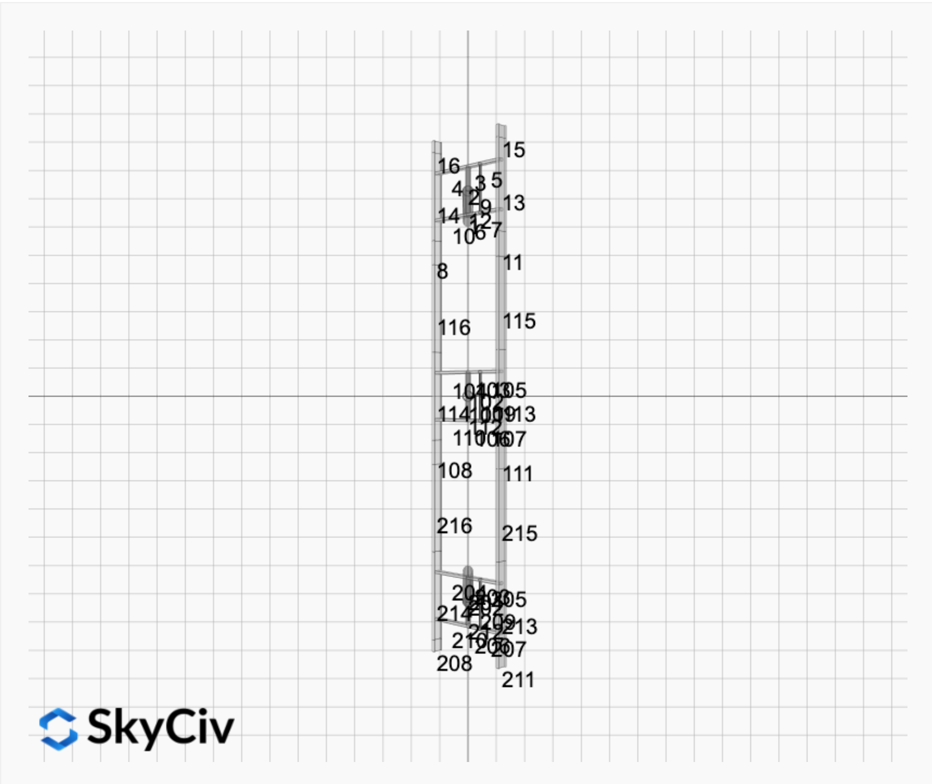
{"wind_speed_override":110,"snow_load_override":68,"direct_snow_load":false,"add_angle_brace":false,"product_type":"Beam","designer_name":"", "designer_email":"christina@mtsolar.us","designer_phone":"","project_id":"W-12514-Guffey 10k - V1Jb","site_address":"PF77+G3, Guffey, CO 80820, USA","module_info":"","module_width":41.1,"module_length":87.2,"number_rows":"4","number_columns":6,"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":45,"ground_clearance":10,"risk_category":"II","exposure_category":"C","frame_duty_override":"XD","pole_override":"10_40","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foundation_size":48,"check_rails": false}

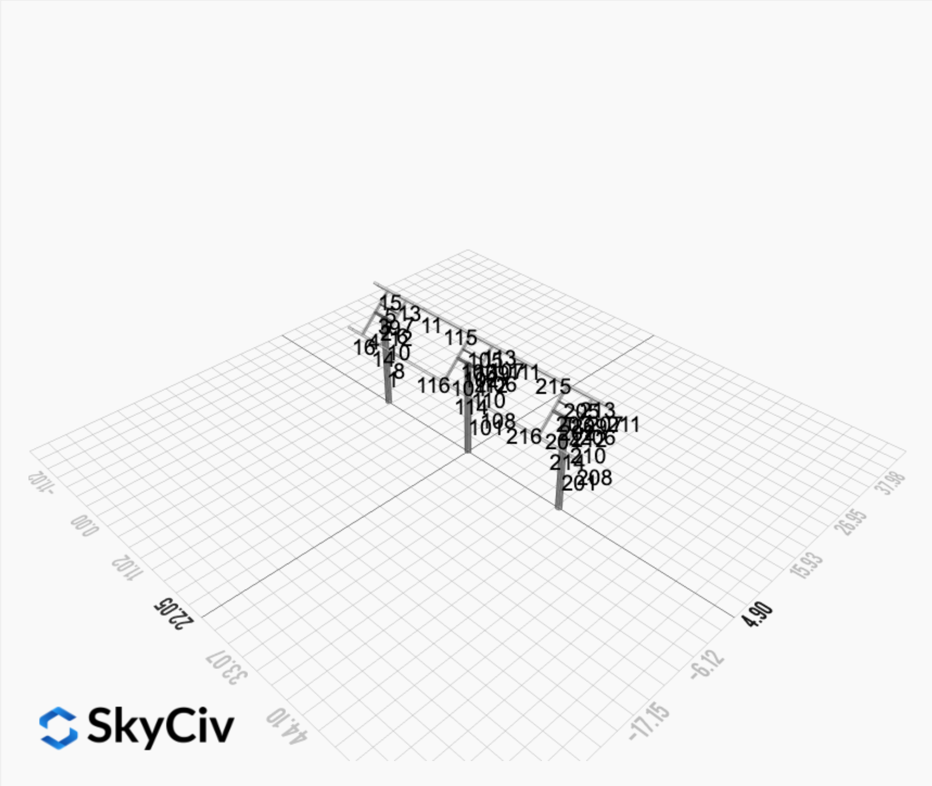
Design Notes:

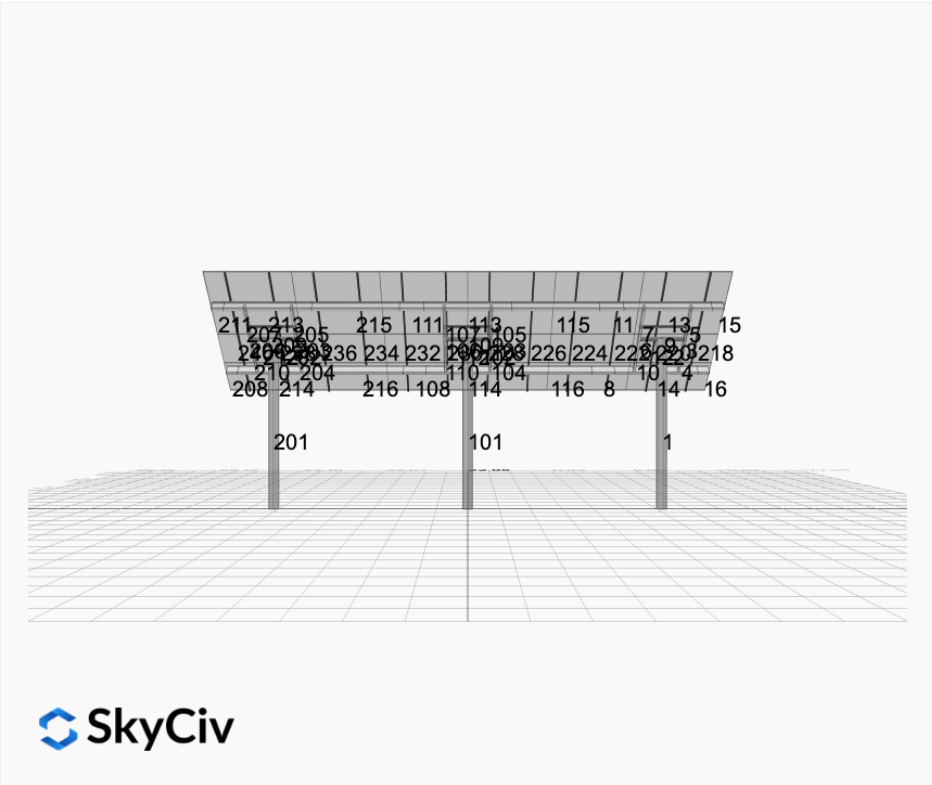
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



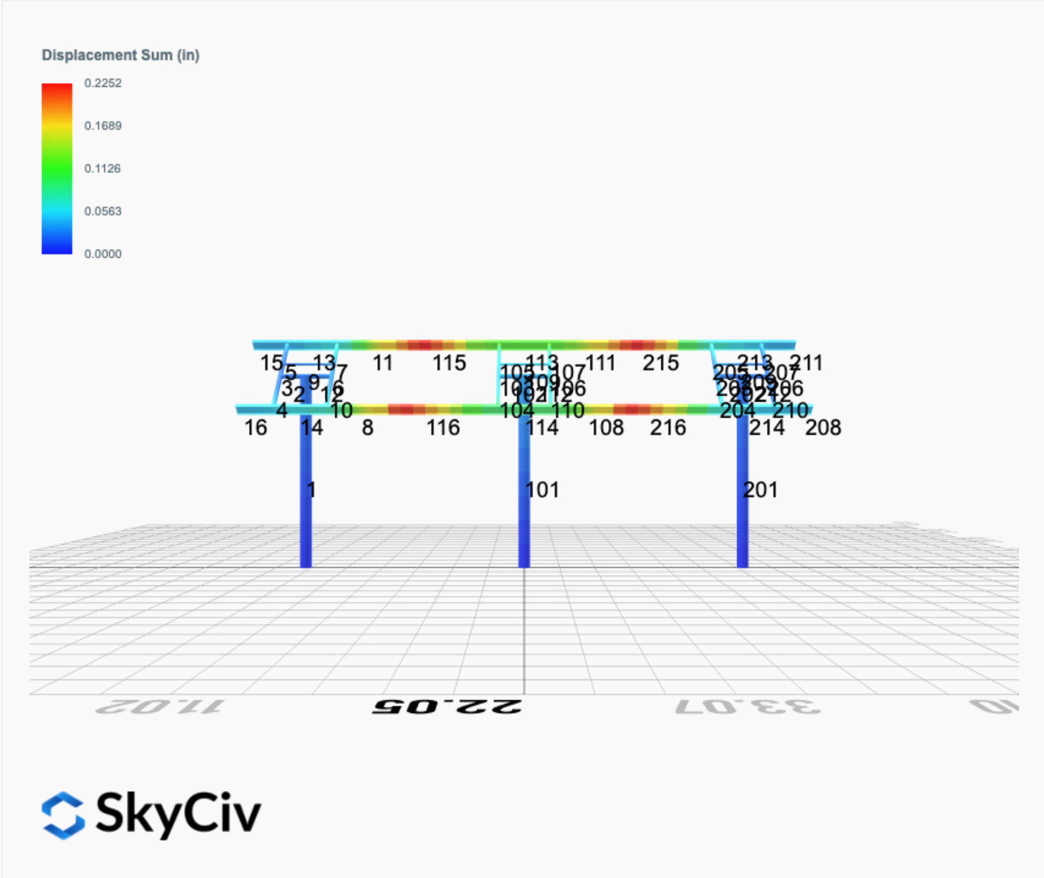


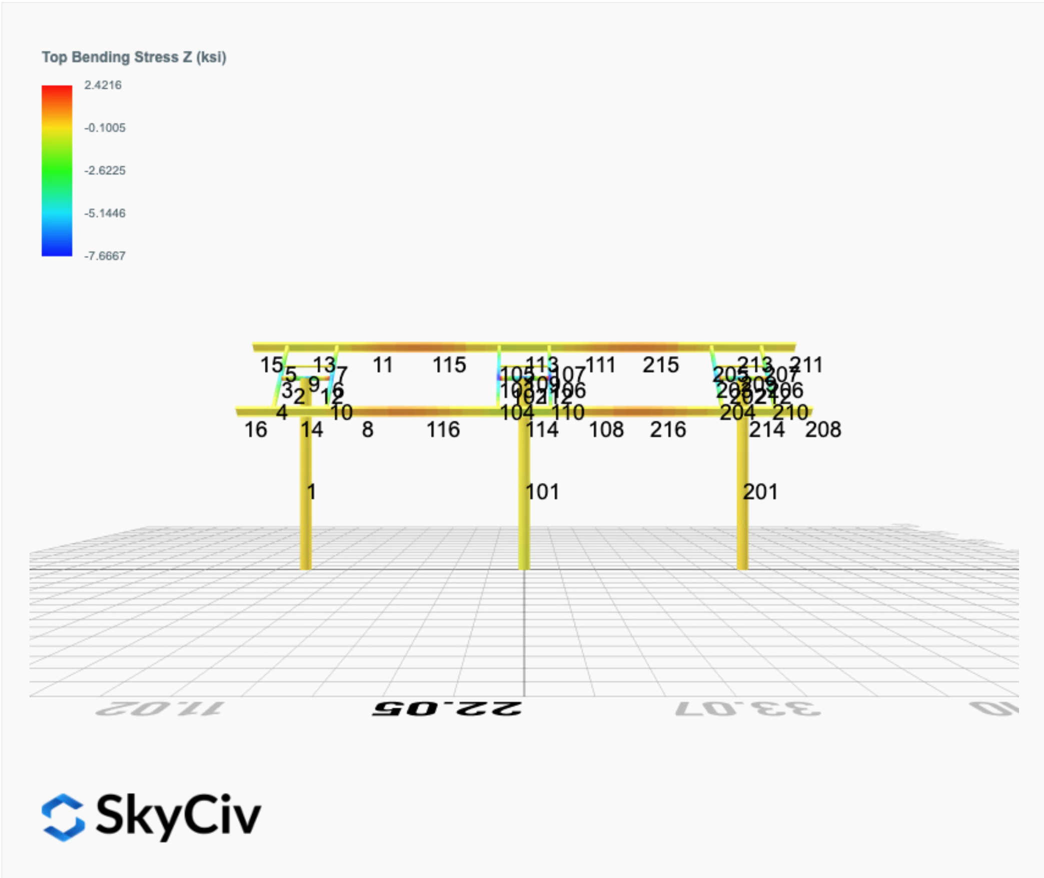


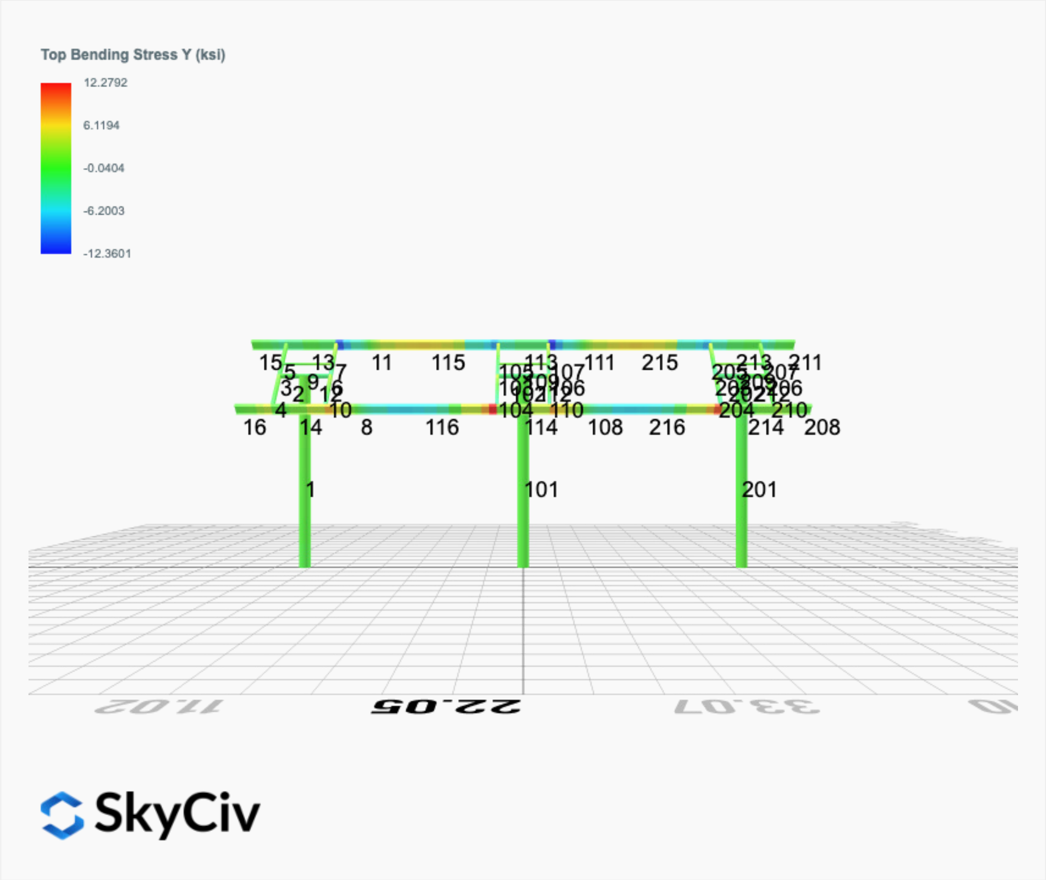

SkyCiv
-11.02
0.00
11.02
22.05
33.07
44.10
37.98
26.95
15.93
4.90
-6.12
-17.15

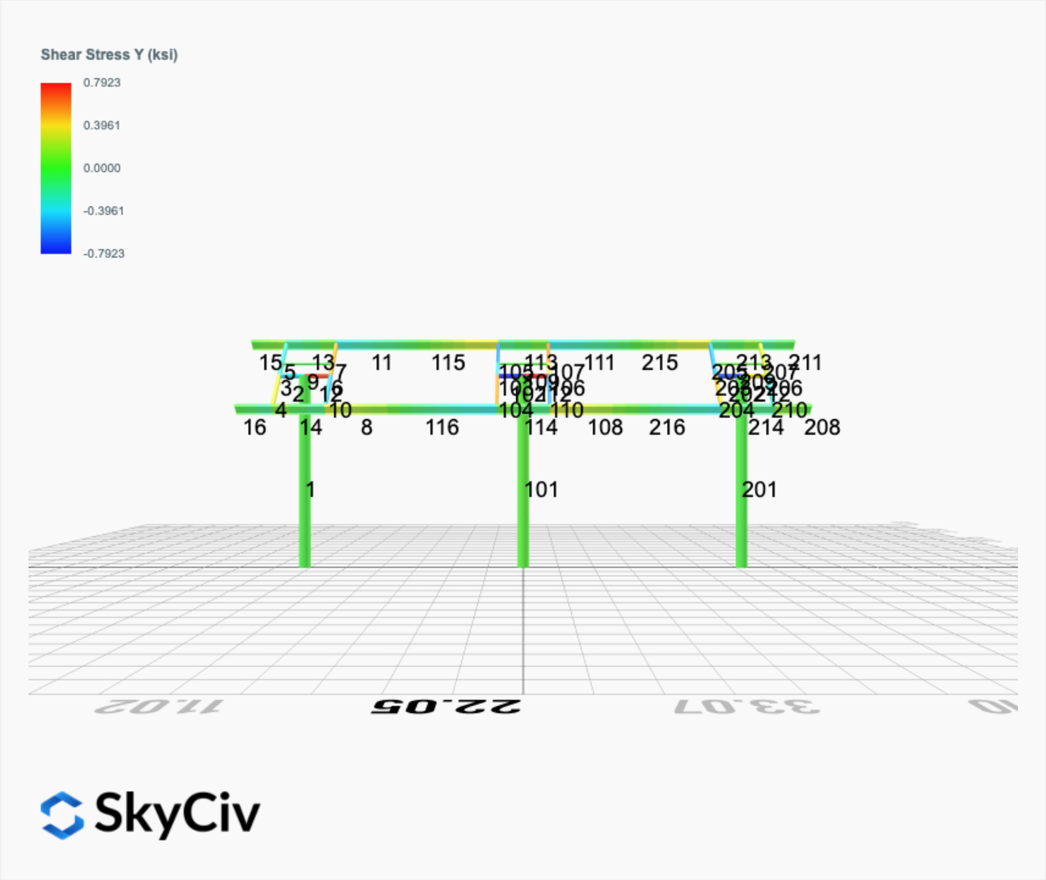

SkyCiv
201
101
1

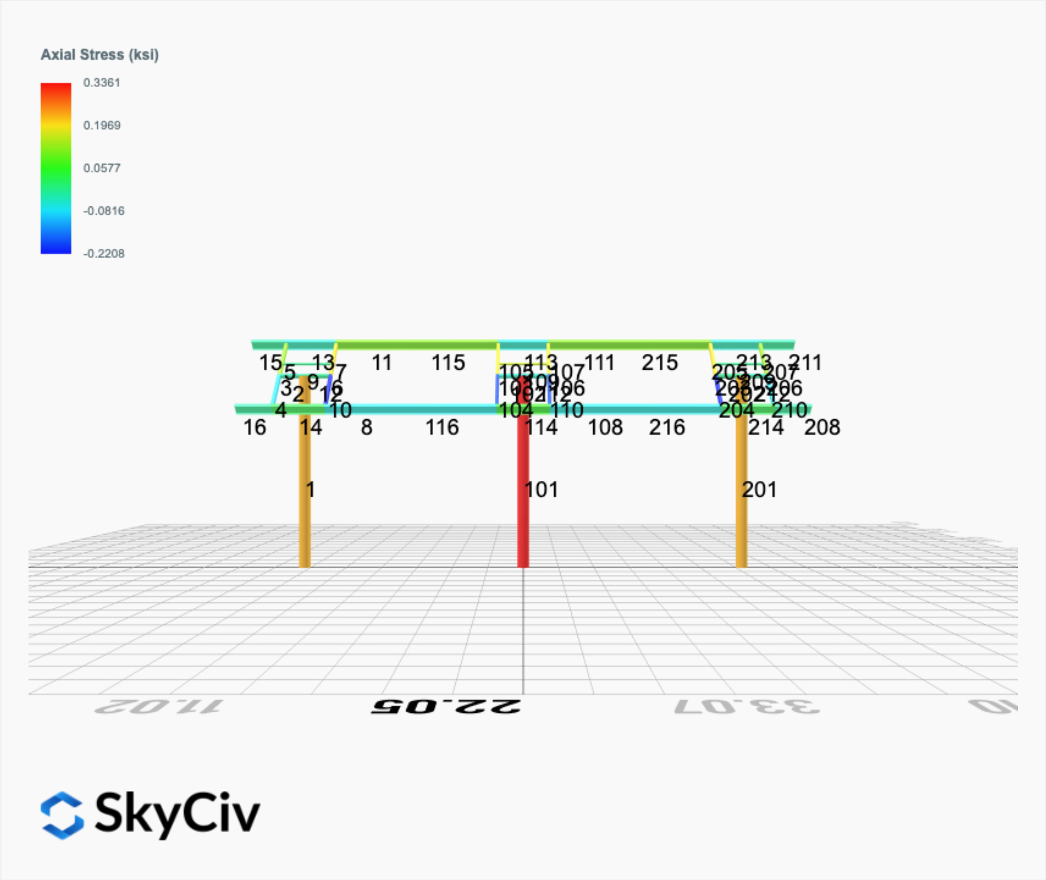
FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
12.2792
6.1194
-0.0404
-6.2003
-12.3601
SkyCiv


Shear Stress Y (ksi)
0.7923
0.3961
0.0000
-0.3961
-0.7923
SkyCiv


Axial Stress (ksi)
0.3361
0.1969
0.0577
-0.0816
-0.2208
1
101
201
11.02
22.05
33.07
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	0.0152	2.0278	0.0438	0.1987	-0.0526	-0.1788
ULS: 2. D + L	0.0152	2.0278	0.0438	0.1987	-0.0526	-0.1788
ULS: 3. D + (S or Lr or R)	0.0547	5.0100	0.1577	0.7162	-0.1910	-0.6950
ULS: 3. D + (S or Lr or R)	0.0152	2.0278	0.0438	0.1987	-0.0526	-0.1788
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0448	4.2644	0.1292	0.5868	-0.1564	-0.5660
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0152	2.0278	0.0438	0.1987	-0.0526	-0.1788
ULS: 5b. D + 0.7E	0.0152	2.0278	0.0438	0.1987	-0.0526	-0.1788
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0448	4.2644	0.1292	0.5868	-0.1564	-0.5660
ULS: 8. 0.6D + 0.7E	0.0091	1.2167	0.0263	0.1192	-0.0316	-0.1073
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5253	4.5016	0.2100	0.9148	-1.0825	39.2140
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5253	4.5016	0.2100	0.9148	-1.0825	39.2140
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8441	0.2443	-0.0723	-0.3011	0.6668	-26.9077
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.6538	0.4380	-0.0691	-0.2871	0.6511	-29.1064
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8606	6.1198	0.2538	1.1238	-0.9288	28.9786
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8606	6.1198	0.2538	1.1238	-0.9288	28.9786
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4165	2.9268	0.0421	0.2119	0.3832	-20.6127
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2738	3.0721	0.0445	0.2224	0.3714	-22.2617
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8901	3.8832	0.1684	0.7358	-0.8250	29.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8901	3.8832	0.1684	0.7358	-0.8250	29.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3869	0.6902	-0.0433	-0.1761	0.4870	-20.2255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2442	0.8354	-0.0409	-0.1656	0.4752	-21.8745
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5314	3.6905	0.1925	0.8353	-1.0614	39.2855
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5314	3.6905	0.1925	0.8353	-1.0614	39.2855
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8380	-0.5668	-0.0898	-0.3806	0.6879	-26.8362
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.6477	-0.3731	-0.0866	-0.3666	0.6721	-29.0349

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	9.2688
Shear X	-4.2342
Shear Z	0.3882
Moment X	1.6989
Moment Y (Twist)	1.8583
Moment Z	65.8930

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	6.1198
Shear X	-2.5314
Shear Z	0.2538
Moment X	1.1238
Moment Y (Twist)	1.0825
Moment Z	39.2855

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	-0.0305	2.4221	-0.0000	0.0000	0.0000	0.4297
ULS: 2. D + L	-0.0305	2.4221	-0.0000	0.0000	0.0000	0.4297
ULS: 3. D + (S or Lr or R)	-0.1094	6.4245	0.0000	-0.0000	0.0000	1.5005
ULS: 3. D + (S or Lr or R)	-0.0305	2.4221	-0.0000	0.0000	0.0000	0.4297
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0896	5.4239	0.0000	0.0000	0.0000	1.2328

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0305	2.4221	-0.0000	0.0000	0.0000	0.4297
ULS: 5b. D + 0.7E	-0.0305	2.4221	-0.0000	0.0000	0.0000	0.4297
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0896	5.4239	0.0000	0.0000	0.0000	1.2328
ULS: 8. 0.6D + 0.7E	-0.0183	1.4532	-0.0000	0.0000	0.0000	0.2578
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.1183	5.6432	-0.0000	0.0000	0.0000	47.4838
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.1183	5.6432	-0.0000	0.0000	0.0000	47.4838
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2211	0.0796	-0.0000	0.0000	0.0000	-31.6921
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.9065	0.3875	-0.0000	0.0000	0.0000	-33.5055
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4055	7.8397	0.0000	0.0000	0.0000	36.5234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4055	7.8397	0.0000	0.0000	0.0000	36.5234
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5990	3.6671	0.0000	0.0000	0.0000	-22.8585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3631	3.8980	0.0000	0.0000	0.0000	-24.2186
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.3463	4.8379	-0.0000	0.0000	0.0000	35.7203
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.3463	4.8379	-0.0000	0.0000	0.0000	35.7203
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6582	0.6652	-0.0000	0.0000	0.0000	-23.6616
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.4222	0.8962	-0.0000	0.0000	0.0000	-25.0217
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.1061	4.6744	-0.0000	0.0000	0.0000	47.3120
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.1061	4.6744	-0.0000	0.0000	0.0000	47.3120
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2333	-0.8892	-0.0000	0.0000	0.0000	-31.8639
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.9187	-0.5813	-0.0000	0.0000	0.0000	-33.6773

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	11.9897
Shear X	-5.2141
Shear Z	0.0000
Moment X	0.0001
Moment Y (Twist)	0.0002
Moment Z	80.3095

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	7.8397
Shear X	-3.1183
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	47.4838

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	0.0152	2.0278	-0.0438	-0.1987	0.0526	-0.1787
ULS: 2. D + L	0.0152	2.0278	-0.0438	-0.1987	0.0526	-0.1787
ULS: 3. D + (S or Lr or R)	0.0547	5.0100	-0.1577	-0.7162	0.1911	-0.6950
ULS: 3. D + (S or Lr or R)	0.0152	2.0278	-0.0438	-0.1987	0.0526	-0.1787
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0448	4.2644	-0.1292	-0.5868	0.1564	-0.5659
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0152	2.0278	-0.0438	-0.1987	0.0526	-0.1787
ULS: 5b. D + 0.7E	0.0152	2.0278	-0.0438	-0.1987	0.0526	-0.1787
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0448	4.2644	-0.1292	-0.5868	0.1564	-0.5659
ULS: 8. 0.6D + 0.7E	0.0091	1.2167	-0.0263	-0.1192	0.0316	-0.1072
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5253	4.5016	-0.2100	-0.9148	1.0825	39.2140
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.5253	4.5016	-0.2100	-0.9148	1.0825	39.2140
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8441	0.2443	0.0723	0.3011	-0.6668	-26.9077
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.6538	0.4380	0.0691	0.2871	-0.6511	-29.1064

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	-1.8606	6.1198	-0.2538	-1.1239	0.9289	28.9786
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8606	6.1198	-0.2538	-1.1239	0.9289	28.9786
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.4165	2.9268	-0.0421	-0.2120	-0.3831	-20.6126
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2738	3.0721	-0.0445	-0.2224	-0.3713	-22.2617
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8901	3.8832	-0.1684	-0.7358	0.8250	29.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.8901	3.8832	-0.1684	-0.7358	0.8250	29.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3869	0.6902	0.0433	0.1761	-0.4870	-20.2255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.2442	0.8354	0.0409	0.1657	-0.4752	-21.8745
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5314	3.6905	-0.1925	-0.8353	1.0614	39.2855
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.5314	3.6905	-0.1925	-0.8353	1.0614	39.2855
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8380	-0.5668	0.0898	0.3806	-0.6879	-26.8362
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.6477	-0.3731	0.0866	0.3666	-0.6721	-29.0349

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	9.2688
Shear X	-4.2342
Shear Z	-0.3882
Moment X	-1.6990
Moment Y (Twist)	1.8583
Moment Z	65.8940

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	6.1198
Shear X	-2.5314
Shear Z	-0.2538
Moment X	-1.1239
Moment Y (Twist)	1.0825
Moment Z	39.2855

Project Details

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

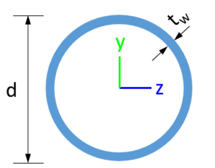
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

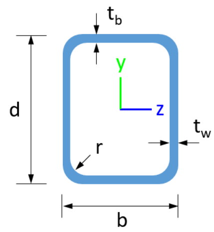
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

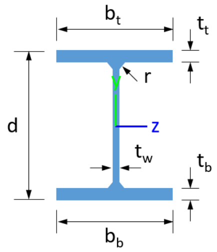
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
11	10in Pipe Sch 40	10.75	0.36					



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



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

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

3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38
17	HSS5x3x1/4	3.37	11.00	4.81	10.70	0.93	3.77	5.38
20	W10x12	3.54	0.05	2.18	53.80	50.90	1.74	12.60

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	11	31.30	31.30	14.90	-	300	200	1
2	6	1.30	1.30	2.00	-	300	200	1
3	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.17,1.17,1.16,1.16,1.17,1.17,1.16,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.14,1.16,1.17,1.17,1.16,1.17	300	200	1
4	17	2.44	2.44	3.75	1.71,1.69,1.71,1.68,1.70,1.71,1.67,1.67,1.65,1.73,1.67,1.67,1.65,1.55,1.67,1.67,1.69,1.68,1.68,1.68,1.63,1.79,1.67,1.67,1.66,1.62	300	200	1
5	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.68,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
6	17	0.92	0.92	1.42	1.20,1.19,1.20,1.19,1.19,1.20,1.18,1.18,1.18,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.19,1.19,1.19,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
7	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
8	20	1.33	1.33	2.05	1.21,1.21,1.21,1.21,1.21,1.21,1.22,1.22,1.22,1.85,1.22,1.22,1.22,1.19,1.21,1.21,1.20,1.22,1.22,1.22,1.22,1.58,1.22,1.22,1.22,1.20	300	200	1
9	3	2.60	2.60	4.00	-	300	200	1
10	17	2.44	2.44	3.75	1.70,1.68,1.70,1.67,1.69,1.70,1.67,1.67,1.66,2.97,1.67,1.67,1.66,1.64,1.67,1.67,1.68,1.68,1.68,1.68,1.64,2.57,1.67,1.67,1.66,1.65	300	200	1
11	20	1.33	1.33	2.05	1.23,1.23,1.23,1.23,1.23,1.23,1.25,1.25,1.28,1.28,1.26,1.26,1.27,1.27,1.24,1.24,1.20,1.15,1.25,1.25,1.28,1.28,1.26,1.26,1.26,1.27	300	200	1
12	6	1.30	1.30	2.00	-	300	200	1
13	20	4.88	4.00	7.50	1.35,1.36,1.35,1.36,1.36,1.35,1.34,1.34,1.27,1.44,1.34,1.34,1.30,1.42,1.35,1.35,1.45,1.41,1.34,1.34,1.27,1.44,1.33,1.33,1.30,1.42	300	200	1
14	20	4.88	4.00	7.50	1.36,1.39,1.36,1.41,1.38,1.36,1.44,1.44,1.55,1.53,1.44,1.44,1.51,1.40,1.43,1.43,1.36,1.47,1.43,1.43,1.58,1.55,1.44,1.44,1.50,1.34	300	200	1
15	20	2.10	2.10	1.00	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	20	2.10	2.10	1.00	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	11	31.30	31.30	14.90	-	300	200	1
102	6	1.30	1.30	2.00	-	300	200	1
103	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
104	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.68,1.67,1.67,1.64,1.81,1.67,1.67,1.66,1.65	300	200	1
105	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
106	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.17,1.18	300	200	1
107	17	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
108	20	1.33	1.33	2.05	1.43,1.44,1.43,1.44,1.44,1.43,1.32,1.32,1.22,1.31,1.30,1.30,1.27,1.16,1.37,1.37,1.78,1.52,1.32,1.32,1.23,1.24,1.30,1.30,1.28,1.20	300	200	1
109	3	2.60	2.60	4.00	-	300	200	1
110	17	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.74,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.67,1.67,1.67,1.64,1.81,1.67,1.67,1.66,1.65	300	200	1
111	20	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.19,1.19,1.09,1.12,1.18,1.18,1.14,1.15,1.24,1.24,1.89,2.24,1.19,1.19,1.09,1.13,1.18,1.18,1.15,1.16	300	200	1
112	6	1.30	1.30	2.00	-	300	200	1

113	20	4.88	4.00	7.50	1.08,1.08,1.08,1.08,1.08,1.08,2.39,2.39,1.82,2.17,3.01,3.01,3.19,3.34,1.20,1.20,1.04,1.03,2.32,2.32,1.87,2.22,3.22,3.22,3.80,3.79	300	200	1
114	20	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.08,1.64,1.34,1.08,1.08,1.11,1.82,1.07,1.07,1.04,1.06,1.08,1.08,1.57,1.35,1.09,1.09,1.10,1.28	300	200	1
115	20	4.84	4.84	7.45	1.10,1.10,1.10,1.10,1.10,1.10,1.06,1.06,1.04,1.04,1.06,1.06,1.04,1.05,1.08,1.08,1.20,1.42,1.06,1.06,1.04,1.04,1.06,1.06,1.04,1.05	300	200	1
116	20	4.84	4.84	7.45	1.13,1.13,1.13,1.13,1.13,1.13,1.10,1.10,1.07,2.67,1.09,1.09,1.09,1.05,1.11,1.11,1.19,1.14,1.10,1.10,1.07,2.22,1.09,1.09,1.09,1.07	300	200	1
201	11	31.30	31.30	14.90	-	300	200	1
202	6	1.30	1.30	2.00	-	300	200	1
203	17	0.92	0.92	1.42	1.20,1.19,1.20,1.19,1.19,1.20,1.18,1.18,1.18,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.19,1.19,1.19,1.19,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
204	17	2.44	2.44	3.75	1.70,1.68,1.70,1.68,1.69,1.70,1.67,1.67,1.66,2.97,1.67,1.67,1.66,1.64,1.67,1.67,1.68,1.68,1.68,1.68,1.64,2.57,1.67,1.67,1.66,1.65	300	200	1
205	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66	300	200	1
206	17	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.17,1.17,1.16,1.16,1.17,1.17,1.16,1.17,1.17,1.17,1.18,1.19,1.18,1.18,1.14,1.16,1.17,1.17,1.16,1.17	300	200	1
207	17	1.52	1.52	2.33	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.68,1.68,1.67,1.67,1.64,1.65,1.67,1.67,1.66,1.66	300	200	1
208	20	2.10	2.10	1.00	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	3	2.60	2.60	4.00	-	300	200	1
210	17	2.44	2.44	3.75	1.71,1.69,1.71,1.68,1.70,1.71,1.67,1.67,1.65,1.73,1.67,1.67,1.65,1.55,1.67,1.67,1.69,1.68,1.68,1.68,1.63,1.79,1.67,1.67,1.66,1.62	300	200	1
211	20	2.10	2.10	1.00	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	6	1.30	1.30	2.00	-	300	200	1
213	20	4.88	4.00	7.50	1.35,1.36,1.35,1.36,1.36,1.35,1.34,1.34,1.26,1.44,1.33,1.33,1.30,1.42,1.35,1.35,1.45,1.41,1.34,1.34,1.27,1.44,1.33,1.33,1.30,1.42	300	200	1
214	20	4.88	4.00	7.50	1.36,1.39,1.36,1.41,1.38,1.36,1.44,1.44,1.55,1.53,1.44,1.44,1.51,1.40,1.43,1.43,1.36,1.47,1.43,1.43,1.58,1.55,1.44,1.44,1.50,1.33	300	200	1
215	20	4.84	4.84	7.45	1.07,1.07,1.07,1.07,1.07,1.07,1.08,1.08,1.13,1.11,1.08,1.08,1.10,1.10,1.08,1.08,1.07,1.08,1.08,1.08,1.12,1.11,1.09,1.09,1.10,1.09	300	200	1
216	20	4.84	4.84	7.45	1.06,1.06,1.06,1.06,1.06,1.06,1.07,1.07,1.07,2.85,1.07,1.07,1.07,1.07,1.06,1.06,1.06,1.06,1.07,1.07,1.07,2.59,1.07,1.07,1.07,1.07	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	535.87	249.61	147.68	147.68	160.76	160.76
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	38.68	6.46	56.26	44.91
14	159.30	97.43	40.82	6.46	56.26	44.91
15	159.30	137.23	46.90	6.46	56.26	44.91
16	159.30	137.23	46.90	6.46	56.26	44.91
101	535.87	249.61	147.68	147.68	160.76	160.76

101	535.87	249.61	147.68	147.68	160.76	160.76
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.41	6.46	56.26	44.91
114	159.30	97.43	31.91	6.46	56.26	44.91
115	159.30	97.82	31.86	6.46	56.26	44.91
116	159.30	97.82	32.40	6.46	56.26	44.91
201	535.87	249.61	147.68	147.68	160.76	160.76
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	137.23	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	137.23	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	38.65	6.46	56.26	44.91
214	159.30	97.43	40.79	6.46	56.26	44.91
215	159.30	97.82	32.74	6.46	56.26	44.91
216	159.30	97.82	32.58	6.46	56.26	44.91

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.037	0.446	0.028	0.026	0.002	0.474	#13	0.511	Not Required	Pass
2	0.001	0.168	0.112	0.043	0.024	0.254	#13	0.054	Not Required	Pass
3	0.006	0.321	0.030	0.031	0.008	0.338	#13	0.046	Not Required	Pass
4	0.005	0.282	0.052	0.028	0.012	0.325	#21	0.082	Not Required	Pass
5	0.005	0.199	0.027	0.032	0.007	0.203	#13	0.076	Not Required	Pass
6	0.010	0.477	0.096	0.049	0.030	0.527	#13	0.046	Not Required	Pass
7	0.010	0.296	0.125	0.047	0.030	0.306	#13	0.076	Not Required	Pass
8	0.003	0.084	0.116	0.023	0.018	0.194	#21	0.102	Not Required	Pass
9	0.002	0.041	0.062	0.003	0.003	0.093	#13	0.206	Not Required	Pass
10	0.010	0.413	0.120	0.041	0.027	0.457	#21	0.082	Not Required	Pass
11	0.005	0.096	0.114	0.028	0.018	0.200	#21	0.102	Not Required	Pass
12	0.001	0.312	0.180	0.071	0.033	0.473	#13	0.054	Not Required	Pass
13	0.007	0.052	0.357	0.038	0.024	0.364	#23	0.306	Not Required	Pass
14	0.004	0.044	0.353	0.031	0.024	0.363	#24	0.204	Not Required	Pass
15	0.000	0.003	0.013	0.006	0.004	0.016	#21	Not Required	Not Required	Pass

16	0.000	0.003	0.013	0.005	0.004	0.016	#21	Not Required	Not Required	Pass
101	0.048	0.544	0.000	0.032	0.000	0.564	#13	0.511	Not Required	Pass
102	0.002	0.316	0.185	0.075	0.035	0.466	#13	0.054	Not Required	Pass
103	0.010	0.489	0.071	0.049	0.017	0.533	#21	0.046	Not Required	Pass
104	0.010	0.472	0.134	0.047	0.030	0.554	#21	0.082	Not Required	Pass
105	0.010	0.303	0.137	0.048	0.035	0.322	#13	0.076	Not Required	Pass
106	0.010	0.489	0.071	0.049	0.017	0.533	#21	0.046	Not Required	Pass
107	0.010	0.303	0.137	0.048	0.035	0.322	#13	0.076	Not Required	Pass
108	0.003	0.074	0.118	0.026	0.018	0.155	#21	0.102	Not Required	Pass
109	0.009	0.028	0.036	0.001	0.000	0.064	#21	0.206	Not Required	Pass
110	0.010	0.472	0.134	0.047	0.030	0.554	#21	0.082	Not Required	Pass
111	0.005	0.104	0.120	0.025	0.018	0.175	#21	0.102	Not Required	Pass
112	0.002	0.316	0.185	0.075	0.035	0.466	#13	0.054	Not Required	Pass
113	0.007	0.056	0.383	0.035	0.025	0.415	#21	0.306	Not Required	Pass
114	0.006	0.083	0.381	0.034	0.025	0.446	#21	0.306	Not Required	Pass
115	0.007	0.179	0.198	0.025	0.018	0.350	#21	0.370	Not Required	Pass
116	0.003	0.140	0.196	0.026	0.018	0.324	#21	0.370	Not Required	Pass
201	0.037	0.446	0.028	0.026	0.002	0.474	#13	0.511	Not Required	Pass
202	0.001	0.312	0.180	0.071	0.033	0.473	#13	0.054	Not Required	Pass
203	0.010	0.477	0.096	0.049	0.030	0.527	#13	0.046	Not Required	Pass
204	0.010	0.413	0.120	0.041	0.027	0.457	#21	0.082	Not Required	Pass
205	0.010	0.296	0.125	0.047	0.030	0.306	#13	0.076	Not Required	Pass
206	0.006	0.321	0.030	0.031	0.008	0.338	#13	0.046	Not Required	Pass
207	0.005	0.199	0.027	0.032	0.007	0.203	#13	0.076	Not Required	Pass
208	0.000	0.003	0.013	0.005	0.004	0.016	#21	Not Required	Not Required	Pass
209	0.002	0.041	0.062	0.003	0.003	0.093	#13	0.206	Not Required	Pass
210	0.005	0.282	0.052	0.028	0.012	0.325	#21	0.082	Not Required	Pass
211	0.000	0.003	0.013	0.006	0.004	0.016	#21	Not Required	Not Required	Pass
212	0.001	0.168	0.112	0.043	0.024	0.254	#13	0.054	Not Required	Pass
213	0.007	0.052	0.357	0.038	0.024	0.364	#23	0.204	Not Required	Pass
214	0.004	0.044	0.353	0.031	0.024	0.363	#24	0.306	Not Required	Pass
215	0.007	0.175	0.197	0.028	0.018	0.350	#21	0.370	Not Required	Pass
216	0.003	0.145	0.197	0.023	0.018	0.331	#21	0.370	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

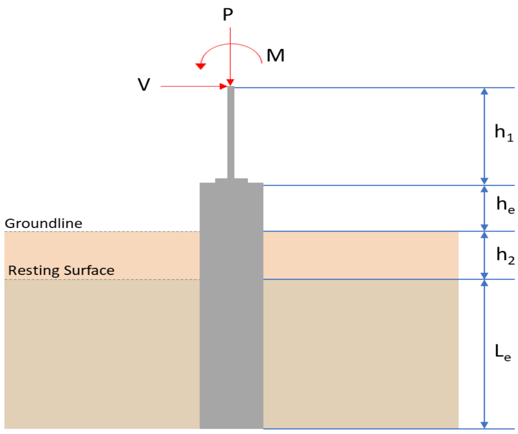
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48\ \text{in}$ - Pile width

$D = 48\ \text{in}$ - Pile depth

$L = 7.5\ \text{ft}$ - 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)	6.120	9.269
V_x (kip)	-2.531	-4.234
V_z (kip)	0.254	0.388
M_x (kipft)	1.124	1.699
M_z (kipft)	39.285	65.893

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,

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

$$H_o = \frac{(-2.531\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.40303\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.285\ \text{kipft}) + ((-2.531\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 6.2556\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.9303\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.254\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.040446\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.124\ \text{kipft}) + ((0.254\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.17898\ \text{kipft/ft}$$

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:
$L_{e,z} = 2.7596\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.9303\ \text{ft}), (2.7596\ \text{ft})]$$

$$L_{e,req} = 6.93\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 7.5\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.93\ \text{ft})}{(7.5\ \text{ft})}$$

$$Ratio = 0.924$$

Status: **PASS**
Ratio: **0.920**

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

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

$$q = \frac{(6.12\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.3825\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.3825\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.19125$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.40303\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.2556\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2556\ \text{kipft/ft}) \times (7.5\ \text{ft})) + \left(3 \times (-0.40303\ \text{kip/ft}) \times (7.5\ \text{ft})^2\right)}{(6 \times (6.2556\ \text{kipft/ft})) + (4 \times (-0.40303\ \text{kip/ft}) \times (7.5\ \text{ft}))}$$

$$a = 5.1523\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.2556\ \text{kipft/ft})) + (3 \times (-0.40303\ \text{kip/ft}) \times (7.5\ \text{ft}))]^2}{(7.5\ \text{ft})^2 \times [(3 \times (6.2556\ \text{kipft/ft})) + (2 \times (-0.40303\ \text{kip/ft}) \times (7.5\ \text{ft}))]}$$

$$p = 0.26678\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (6.2556\ \text{kipft/ft})) + ((-0.40303\ \text{kip/ft}) \times (7.5\ \text{ft}))]}{(7.5\ \text{ft})^2}$$

$$s = 1.0121\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(5.1523\ \text{ft})}{2}$$

$$p_a = 0.38642\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

	$$Ratio = \frac{p}{p_a}$$ $$Ratio = \frac{\left(0.26678\ \text{kip/ft}^2\right)}{\left(0.38642\ \text{kip/ft}^2\right)}$$ $$Ratio = 0.69039$$	Status: **PASS** Ratio: **0.690**
	p_s - Allowable lateral soil pressure at depth L_e, $$p_s = R\,L_e$$ $$p_s = (150\ \text{psf/ft}) \times (7.5\ \text{ft})$$ $$p_s = 1.125\ \text{kip/ft}^2$$ $Ratio$ - Lateral soil capacity $$Ratio = \frac{s}{p_s}$$ $$Ratio = \frac{\left(1.0121\ \text{kip/ft}^2\right)}{\left(1.125\ \text{kip/ft}^2\right)}$$ $$Ratio = 0.89965$$	Status: **PASS** Ratio: **0.900**

Considering z-direction:

$H_o = 0.040446\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.17898\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17898\ \text{kipft/ft}) \times (7.5\ \text{ft})) + \left(3 \times (0.040446\ \text{kip/ft}) \times (7.5\ \text{ft})^2\right)}{(6 \times (0.17898\ \text{kipft/ft})) + (4 \times (0.040446\ \text{kip/ft}) \times (7.5\ \text{ft}))}$$

$$a = 5.3316\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.17898\ \text{kipft/ft})) + (3 \times (0.040446\ \text{kip/ft}) \times (7.5\ \text{ft}))]^{\,2}}{(7.5\ \text{ft})^2 \times [(3 \times (0.17898\ \text{kipft/ft})) + (2 \times (0.040446\ \text{kip/ft}) \times (7.5\ \text{ft}))]}$$

$$p = 0.030823\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.17898\ \text{kipft/ft})) + ((0.040446\ \text{kip/ft}) \times (7.5\ \text{ft}))]}{(7.5\ \text{ft})^2}$$

$$s = 0.070539\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(5.3316\ \text{ft})}{2}$$

$$p_a = 0.39987\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.030823\ \text{kip/ft}^2\right)}{\left(0.39987\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.077082$$

Status: **PASS**
Ratio: **0.080**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.5\ \mathrm{ft})$$

$$p_s = 1.125\ \mathrm{kip/ft^2}$$

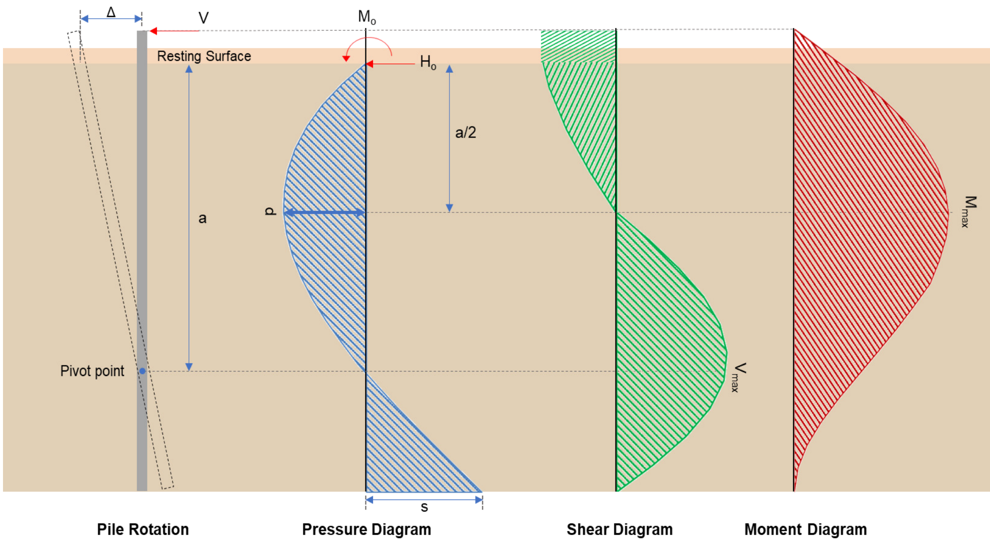
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.070539\ \mathrm{kip/ft^2})}{(1.125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.062702$$

Status: **PASS**
Ratio: **0.060**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.234\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.6742\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(65.893\ \mathrm{kipft}) + ((-4.234\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 10.493\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.493\ \mathrm{kipft/ft})}{(-0.6742\ \mathrm{kip/ft})}$$

$$E = 15.563\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.493\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.6742\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (10.493\ \text{kipft/ft})) + (4 \times (-0.6742\ \text{kip/ft}) \times (7.5\ \text{ft}))}$$

$$a = 5.152\ \text{ft}$$

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

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

$$V_{max} = ((-0.6742\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.563\ \text{ft})}{(7.5\ \text{ft})} + 3\right) \times \left(\frac{(5.152\ \text{ft})}{(7.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.563\ \text{ft})}{(7.5\ \text{ft})} + 2\right) \times \left(\frac{(5.152\ \text{ft})}{(7.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 11.683\ \text{kip}$$

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

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

$$M_{max} = ((-0.6742\ \text{kip/ft}) \times (48\ \text{in}) \times (7.5\ \text{ft})) \times \left[\left(\frac{(15.563\ \text{ft})}{(7.5\ \text{ft})} + \frac{(5.152\ \text{ft})}{2 \times (7.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (15.563\ \text{ft})}{(7.5\ \text{ft})} + 3\right) \times \left(\frac{(5.152\ \text{ft})}{2 \times (7.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (15.563\ \text{ft})}{(7.5\ \text{ft})} + 2\right) \times \left(\frac{(5.152\ \text{ft})}{2 \times (7.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 41.971\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.388\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.061783\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.699\ \text{kipft}) + ((0.388\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.27054\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.27054\ \text{kipft/ft})}{(0.061783\ \text{kip/ft})}$$

$$E = 4.3789\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.27054\ \text{kipft/ft}) \times (7.5\ \text{ft})) + \left(3 \times (0.061783\ \text{kip/ft}) \times (7.5\ \text{ft})^2\right)}{(6 \times (0.27054\ \text{kipft/ft})) + (4 \times (0.061783\ \text{kip/ft}) \times (7.5\ \text{ft}))}$$

$$a = 5.3332\ \text{ft}$$

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

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

$$V_{max} = ((0.061783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3789 \text{ ft})}{(7.5 \text{ ft})} + 3\right) \times \left(\frac{(5.3332 \text{ ft})}{(7.5 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.3789 \text{ ft})}{(7.5 \text{ ft})} + 2\right) \times \left(\frac{(5.3332 \text{ ft})}{(7.5 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.4196 \text{ kip}$$

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

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

$$M_{max} = ((0.061783 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(4.3789 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3332 \text{ ft})}{2 \times (7.5 \text{ ft})}\right) - \left[\left(\frac{4 \times (4.3789 \text{ ft})}{(7.5 \text{ ft})} + 3\right) \times \left(\frac{(5.3332 \text{ ft})}{2 \times (7.5 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.3789 \text{ ft})}{(7.5 \text{ ft})} + 2\right) \times \left(\frac{(5.3332 \text{ ft})}{2 \times (7.5 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 1.4078 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.269 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.288 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.288 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.269\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0034648$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.269 \text{ kip} \rightarrow 9269 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9269 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.72 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.72 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.72 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.72 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.9 \text{ kip}$$

Considering x-direction:

$V_{max} = 11.683 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(11.683 \text{ kip})}{(110.9 \text{ kip})}$$ $$Ratio = 0.10535$$	Status: **PASS** Ratio: **0.110**
	Considering z-direction: $V_{max} = 0.4196 \text{ kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.4196 \text{ kip})}{(110.9 \text{ kip})}$$ $$Ratio = 0.0037836$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$ $$S_m = 18432 \text{ in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_{ck}\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ 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:** $M_{max} = 41.971 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(41.971 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.16815$$	Status: **PASS** Ratio: **0.170**
	Considering z-direction: $M_{max} = 1.4078 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(1.4078\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.0056403$$	Status: **PASS** Ratio: **0.010**	

REFERENCES	CALCULATIONS	RESULTS

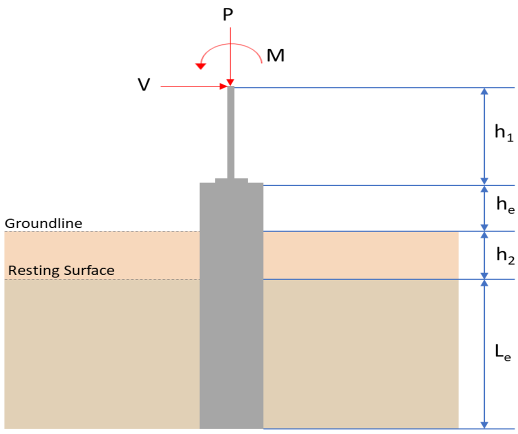
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 7.5$ 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)	6.120	9.269
V_x (kip)	-2.531	-4.234
V_z (kip)	-0.254	-0.388
M_x (kipft)	-1.124	-1.699
M_z (kipft)	39.285	65.894

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,

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

$$H_o = \frac{(-2.531\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.40303\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.285\text{ kipft}) + ((-2.531\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 6.2556\text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.9303\text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.254\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.040446\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.124\text{ kipft}) + ((-0.254\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 0.17898\text{ kipft/ft}$$

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:
$L_{e,z} = 2.0975\text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.9303\text{ ft}), (2.0975\text{ ft})]$$

$$L_{e,req} = 6.93\text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5\text{ ft}) - (0\text{ ft}) - (0\text{ ft})$$

$$L_e = 7.5\text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.93\text{ ft})}{(7.5\text{ ft})}$$

$$Ratio = 0.924$$

Status: **PASS**
Ratio: **0.920**

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

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

$$q = \frac{(6.12\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.3825\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.3825\ \mathrm{kip/ft}^2\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.19125$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.40303\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 6.2556\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2556\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.40303\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{(6 \times (6.2556\ \mathrm{kipft/ft})) + (4 \times (-0.40303\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))}$$

$$a = 5.1523\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.2556\ \mathrm{kipft/ft})) + (3 \times (-0.40303\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]^{\,2}}{(7.5\ \mathrm{ft})^2 \times [(3 \times (6.2556\ \mathrm{kipft/ft})) + (2 \times (-0.40303\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}$$

$$p = 0.26678\ \mathrm{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (6.2556\ \mathrm{kipft/ft})) + ((-0.40303\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}{(7.5\ \mathrm{ft})^2}$$

$$s = 1.0121\ \mathrm{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.1523\ \mathrm{ft})}{2}$$

$$p_a = 0.38642\ \mathrm{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.26678\ \mathrm{kip/ft^2}\right)}{\left(0.38642\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.69039$$

Status: **PASS**
Ratio: **0.690**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.5\ \mathrm{ft})$$

$$p_s = 1.125\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.0121\ \mathrm{kip/ft^2}\right)}{\left(1.125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.89965$$

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.040446\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.17898\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17898\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.040446\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{(6 \times (0.17898\ \mathrm{kipft/ft})) + (4 \times (-0.040446\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))}$$

$$a = 5.3316\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.17898\ \mathrm{kipft/ft})) + (3 \times (-0.040446\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]^{\ 2}}{(7.5\ \mathrm{ft})^2 \times [(3 \times (0.17898\ \mathrm{kipft/ft})) + (2 \times (-0.040446\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}$$

$$p = -0.007203\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.17898\ \mathrm{kipft/ft})) + ((-0.040446\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))]}{(7.5\ \mathrm{ft})^2}$$

$$s = 0.0058259\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.3316\ \mathrm{ft})}{2}$$

$$p_a = 0.39987\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.007203\ \mathrm{kip/ft^2}\right)}{\left(0.39987\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.018013$$

Status: **PASS**
Ratio: **-0.020**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.5\ \mathrm{ft})$$

$$p_s = 1.125\ \mathrm{kip/ft^2}$$

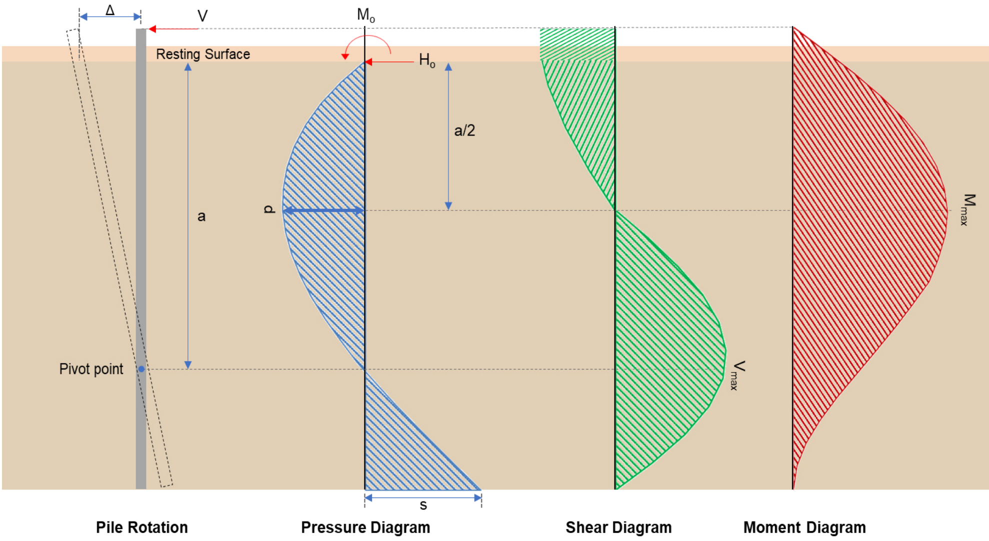
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0058259\ \mathrm{kip/ft^2})}{(1.125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.0051786$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.234\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.6742\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(65.894\ \mathrm{kipft}) + ((-4.234\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 10.493\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.493\ \mathrm{kipft/ft})}{(-0.6742\ \mathrm{kip/ft})}$$

$$E = 15.563\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.493\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.6742\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{\ldots}$$

$$a = \frac{}{(6 \times (10.493\ \mathrm{kipft/ft})) + (4 \times (-0.6742\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))}$$

$$a = 5.152\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.6742\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.563\ \mathrm{ft})}{(7.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.152\ \mathrm{ft})}{(7.5\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.563\ \mathrm{ft})}{(7.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.152\ \mathrm{ft})}{(7.5\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 11.683\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.6742\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (7.5\ \mathrm{ft})) \times \left[\left(\frac{(15.563\ \mathrm{ft})}{(7.5\ \mathrm{ft})} + \frac{(5.152\ \mathrm{ft})}{2 \times (7.5\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (15.563\ \mathrm{ft})}{(7.5\ \mathrm{ft})} + 3\right) \times \left(\frac{(5.152\ \mathrm{ft})}{2 \times (7.5\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (15.563\ \mathrm{ft})}{(7.5\ \mathrm{ft})} + 2\right) \times \left(\frac{(5.152\ \mathrm{ft})}{2 \times (7.5\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 41.972\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.388\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.061783\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.699\ \mathrm{kipft}) + ((-0.388\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.27054\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.27054\ \mathrm{kipft/ft})}{(-0.061783\ \mathrm{kip/ft})}$$

$$E = 4.3789\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.27054\ \mathrm{kipft/ft}) \times (7.5\ \mathrm{ft})) + \left(3 \times (-0.061783\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft})^2\right)}{(6 \times (0.27054\ \mathrm{kipft/ft})) + (4 \times (-0.061783\ \mathrm{kip/ft}) \times (7.5\ \mathrm{ft}))}$$

$$a = 5.3332\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.061783\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3789\ \text{ft})}{(7.5\ \text{ft})} + 3\right) \times \left(\frac{(5.3332\ \text{ft})}{(7.5\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.3789\ \text{ft})}{(7.5\ \text{ft})} + 2\right) \times \left(\frac{(5.3332\ \text{ft})}{(7.5\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.4196\ \text{kip}$$

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

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

$$M_{max} = ((-0.061783\ \text{kip/ft}) \times (48\ \text{in}) \times (7.5\ \text{ft})) \times \left[\left(\frac{(4.3789\ \text{ft})}{(7.5\ \text{ft})} + \frac{(5.3332\ \text{ft})}{2 \times (7.5\ \text{ft})}\right) - \left[\left(\frac{4 \times (4.3789\ \text{ft})}{(7.5\ \text{ft})} + 3\right) \times \left(\frac{(5.3332\ \text{ft})}{2 \times (7.5\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.3789\ \text{ft})}{(7.5\ \text{ft})} + 2\right) \times \left(\frac{(5.3332\ \text{ft})}{2 \times (7.5\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.4078\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.269\ \text{kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.288\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.288\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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. 10⌀: 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

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

$$Ratio = \frac{(9.269\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0034648$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.269 \text{ kip} \rightarrow 9269 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9269 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.72 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.72 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.72 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.72 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.9 \text{ kip}$$

Considering x-direction:

$V_{max} = 11.683 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(11.683 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.10535$$

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

Considering z-direction:

$V_{max} = 0.4196 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.4196 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.0037836$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
M_n shall be the lesser of:
$\phi M_{n,1}$

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

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

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

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

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 41.972 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(41.972 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.16816$$

Status: **PASS**
Ratio: **0.170**

Considering z-direction:

$M_{max} = 1.4078 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(1.4078 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0056403$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

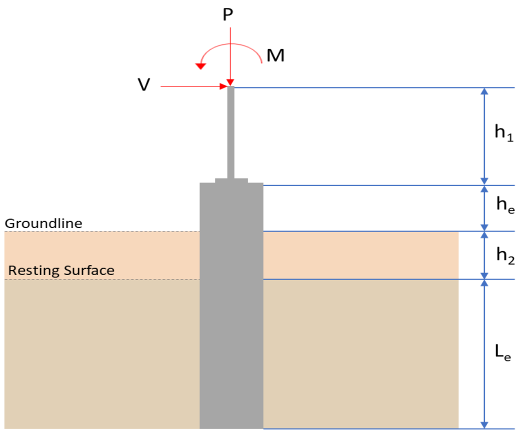
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 7.75$ 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)	7.840	11.990
V_x (kip)	-3.118	-5.214
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	47.484	80.309

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,

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

$$H_o = \frac{(-3.118 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4965 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(47.484 \text{ kipft}) + ((-3.118 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.5611 \text{ kipft/ft}$$

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:
$L_{e,x} = 7.2916 \text{ ft}$ - 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,

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

$$L_{e,req} = MAX[(7.2916 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.292 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(7.292 \text{ ft})}{(7.75 \text{ ft})}$$

$$Ratio = 0.9409$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.84 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.49 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.245$$

Status: **PASS**
Ratio: **0.240**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.9375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.4965\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 7.5611\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.5611\ \mathrm{kipft/ft}) \times (7.75\ \mathrm{ft})) + \left(3 \times (-0.4965\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft})^2\right)}{(6 \times (7.5611\ \mathrm{kipft/ft})) + (4 \times (-0.4965\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))}$$

$$a = 5.3303\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (7.5611\ \mathrm{kipft/ft})) + (3 \times (-0.4965\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]^{\ 2}}{(7.75\ \mathrm{ft})^2 \times [(3 \times (7.5611\ \mathrm{kipft/ft})) + (2 \times (-0.4965\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]}$$

$$p = 0.29138\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (7.5611\ \mathrm{kipft/ft})) + ((-0.4965\ \mathrm{kip/ft}) \times (7.75\ \mathrm{ft}))]}{(7.75\ \mathrm{ft})^2}$$

$$s = 1.1263\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(5.3303\ \mathrm{ft})}{2}$$

$$p_a = 0.39977\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.29138\ \mathrm{kip/ft^2}\right)}{\left(0.39977\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.72886$$

Status: **PASS**
Ratio: **0.730**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7.75\ \mathrm{ft})$$

$$p_s = 1.1625\ \mathrm{kip/ft^2}$$

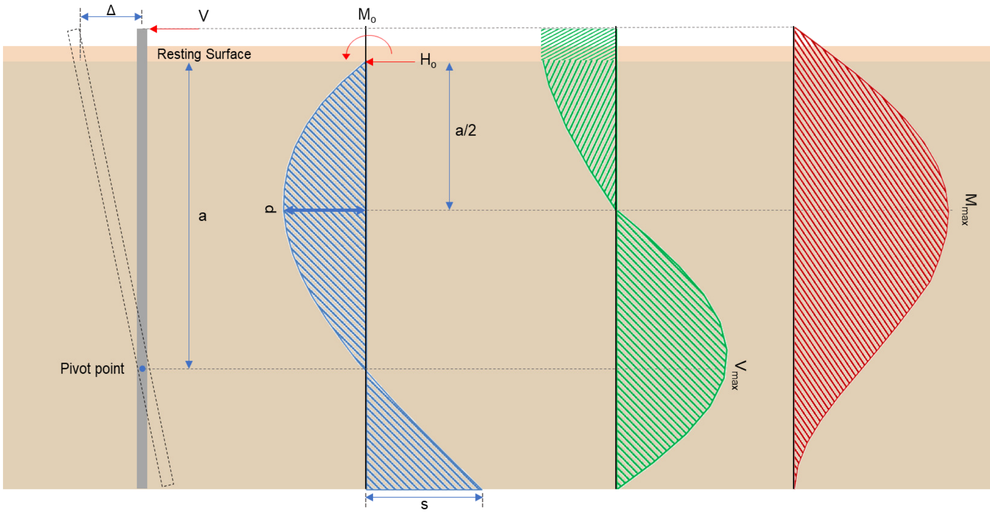
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(1.1263\ \mathrm{kip/ft^2}\right)}{\left(1.1625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.96884$$

Status: PASS
Ratio: 0.970



Pile Rotation **Pressure Diagram** **Shear Diagram** **Moment Diagram**

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.214\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.83025\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(80.309\ \text{kipft}) + ((-5.214\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 12.788\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(12.788\ \text{kipft/ft})}{(-0.83025\ \text{kip/ft})}$$

$$E = 15.403\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (12.788\ \text{kipft/ft}) \times (7.75\ \text{ft})) + \left(3 \times (-0.83025\ \text{kip/ft}) \times (7.75\ \text{ft})^2\right)}{(6 \times (12.788\ \text{kipft/ft})) + (4 \times (-0.83025\ \text{kip/ft}) \times (7.75\ \text{ft}))}$$

$$a = 5.3289\ \text{ft}$$

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

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

$$V_{max} = ((-0.83025\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.403\ \text{ft})}{(7.75\ \text{ft})} + 3\right) \times \left(\frac{(5.3289\ \text{ft})}{(7.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (15.403\ \text{ft})}{(7.75\ \text{ft})} + 2\right) \times \left(\frac{(5.3289\ \text{ft})}{(7.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 13.873\ \text{kip}$$

$$V_{max} = 13.872 \text{ kip}$$

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

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

$$M_{max} = ((-0.83025 \text{ kip/ft}) \times (48 \text{ in}) \times (7.75 \text{ ft})) \times \left[\left(\frac{(15.403 \text{ ft})}{(7.75 \text{ ft})} + \frac{(5.3289 \text{ ft})}{2 \times (7.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (15.403 \text{ ft})}{(7.75 \text{ ft})} + 3\right) \times \left(\frac{(5.3289 \text{ ft})}{2 \times (7.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (15.403 \text{ ft})}{(7.75 \text{ ft})} + 2\right) \times \left(\frac{(5.3289 \text{ ft})}{2 \times (7.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 51.412 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.99 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.198 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.198 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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 25.7.2.1	Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in) s_{ties} - Maximum spacing of ties, $$s_{ties} = Min\left[(16\, d_{bar}), (48\, d_{ties}), Min\left(D, b\right)\right]$$ $$s_{ties} = Min\left[(16 \times (0.625\text{ in})), (48 \times (0.375\text{ in})), Min\left((48\text{ in}), (48\text{ in})\right)\right]$$ $$s_{ties} = 10\text{ in}$$ **Summary:** Main reinforcement: **14 - #5 (0.625 in)** Ties: **#3(0.375 in) - 10 in**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left(\left(60\text{ ksi}\right) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(11.99\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0044819$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\text{ in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $V_{c,max}$ - 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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 11.99\text{ kip} \rightarrow 11990\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11990 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.08 \text{ kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21 \text{ kip}), (120.08 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 120.08 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.08 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.14 \text{ kip}$$

Considering x-direction:

$V_{max} = 13.872 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(13.872 \text{ kip})}{(111.14 \text{ kip})}$$

$$Ratio = 0.12482$$

Status: **PASS**
Ratio: **0.120**

	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:** $M_{max} = 51.412\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(51.412\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.20598$$	Status: **PASS** Ratio: **0.210**