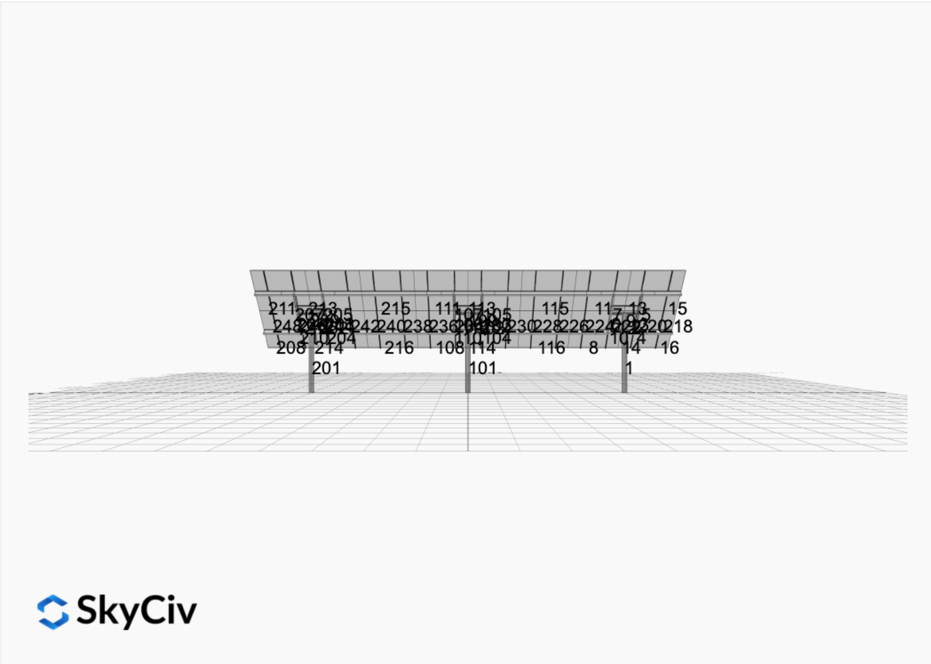
Project Details

Project Name: Vermillion Pivot 4x8-cu
Location: Q6Q5+HW, Greycliff, MT, USA
Unique ID: 3P-22.5-8TOP-HD-45-L-4Hx8W-H43D
Dealer: ______________________________

Date: Tue Jun 10 2025
Number of Modules: 32
Number of Poles: 3
Date Sold: ______________________________



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	60.13 ft
Winter Tilt Angle	46
Front Edge Clearance	6 ft

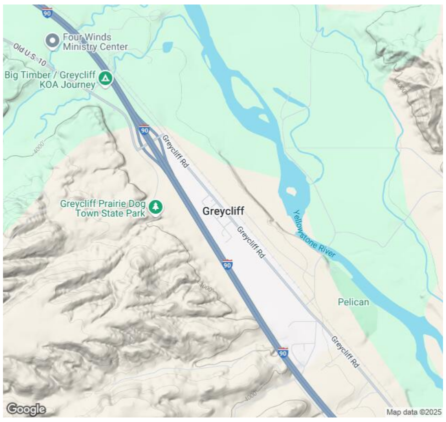
MT Solar Bill of Materials (3P-22.5-8TOP-HD-45-L-4Hx8W-H43D)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	8

Rail Bill of Materials

Part	Qty
Rails (180in)	16
Rail Attachment	32
Module Mid Clamp	48
Module End Clamp	32
Ground Lug	8

Site Details:



Site Address: Q6Q5+HW, Greycliff, MT, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	89.20in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Winter Tilt Angle:	46
Front Edge Clearance:	6
Total Array Height at Tilt:	16.81 ft
Total Frame Length:	60.00 ft
Module Info/Notes:	Solarever SE-182*105-450M-96-BD
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	60.13 ft
Rail Length:	180.40 in
Rail Spacing:	3.76 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	11.41 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 7.25 ft Pile 3: 6.75 ft
Foundation Volume:	12.296 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Q6Q5+HW, Greycliff, MT, USA
Wind Speed:	115 mph
Snow Load:	54 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


```
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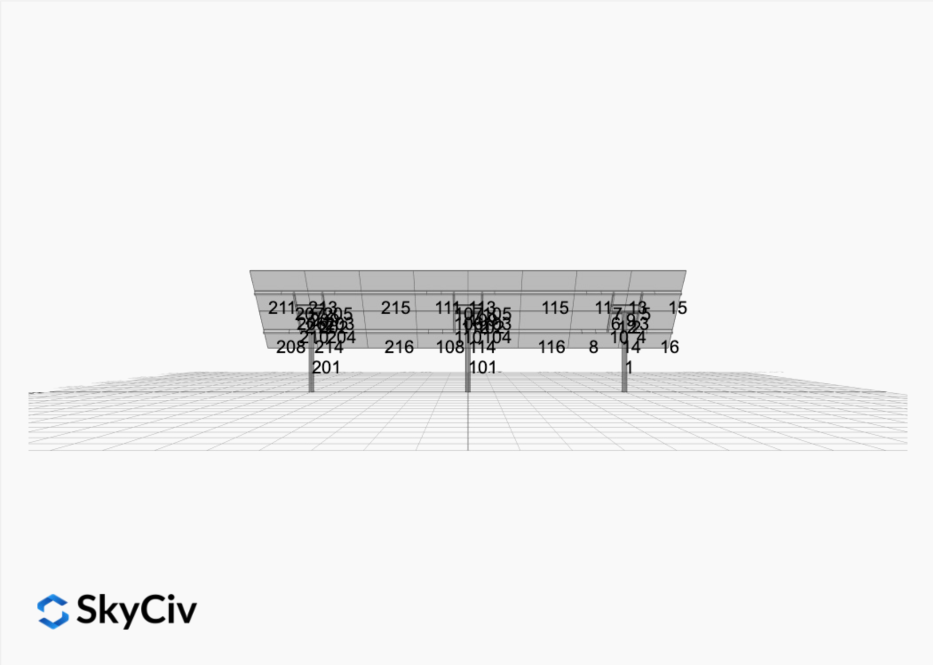
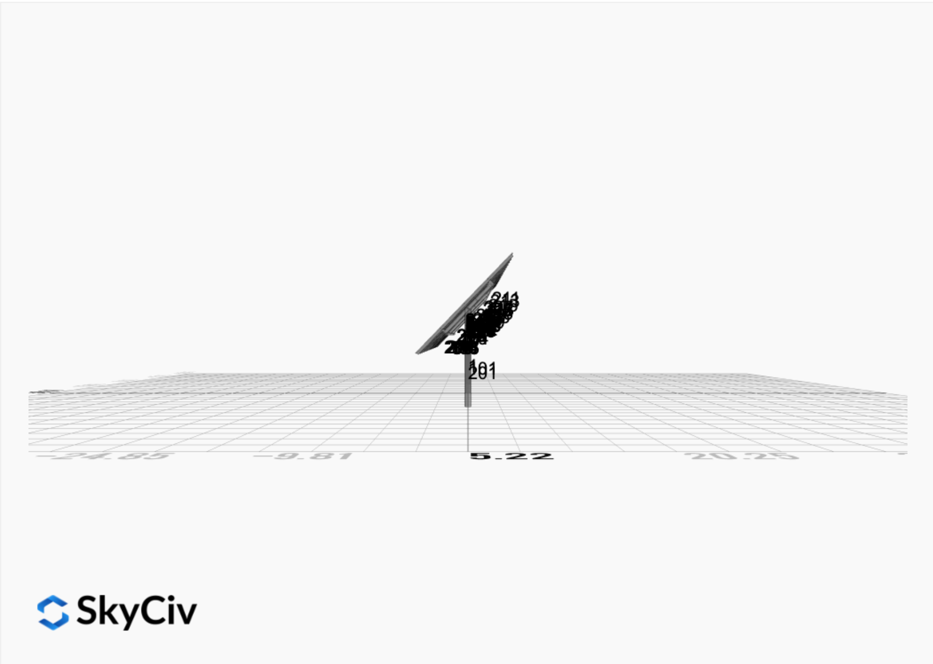
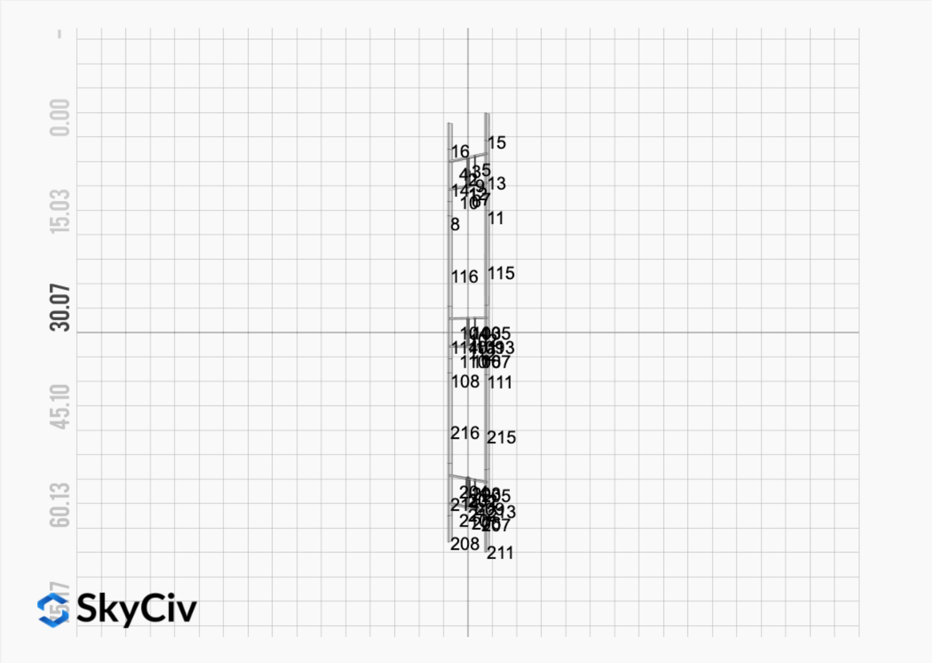
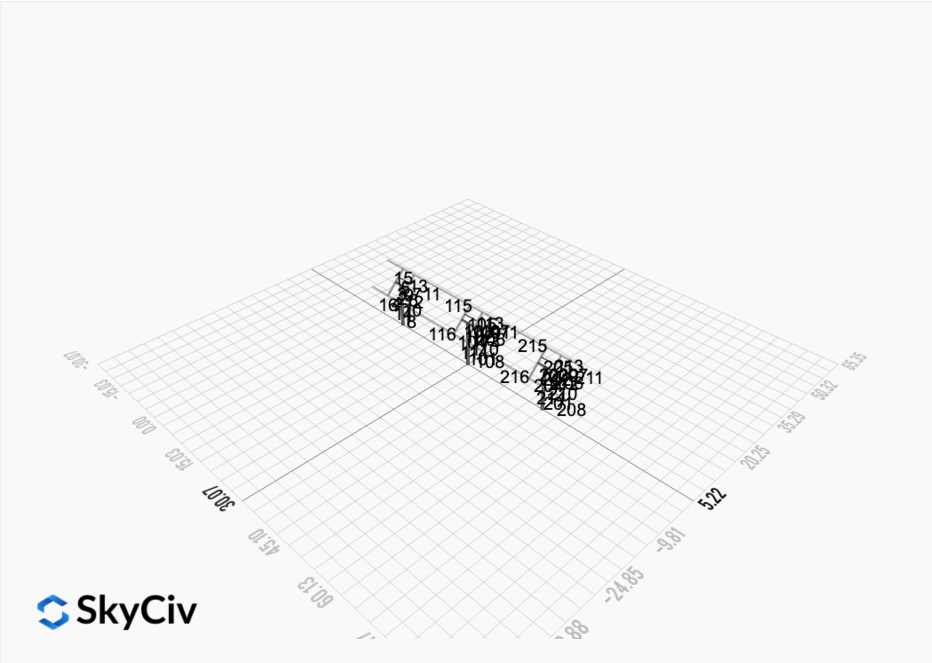
Design Notes:

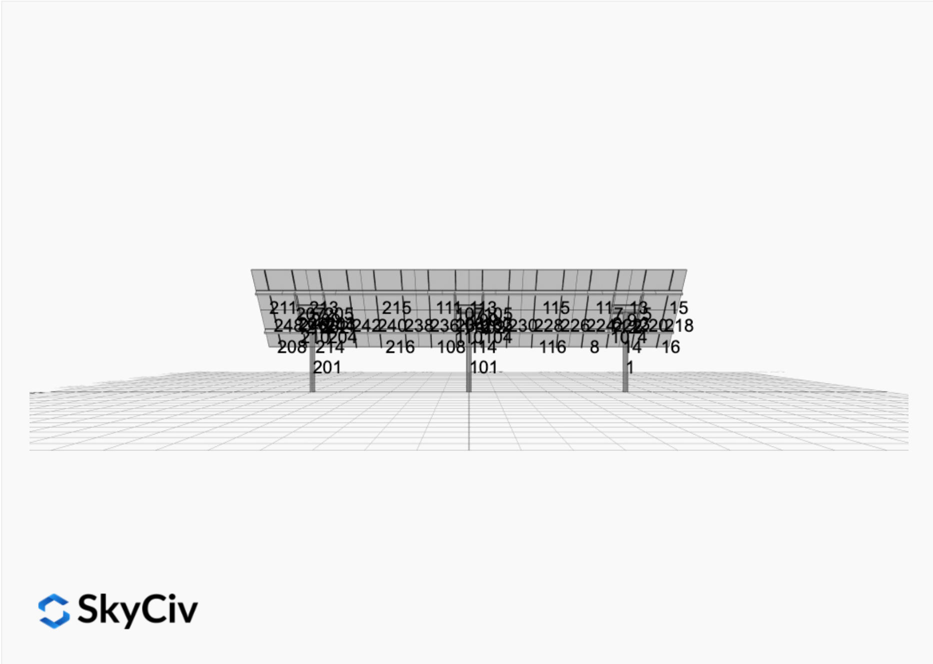
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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







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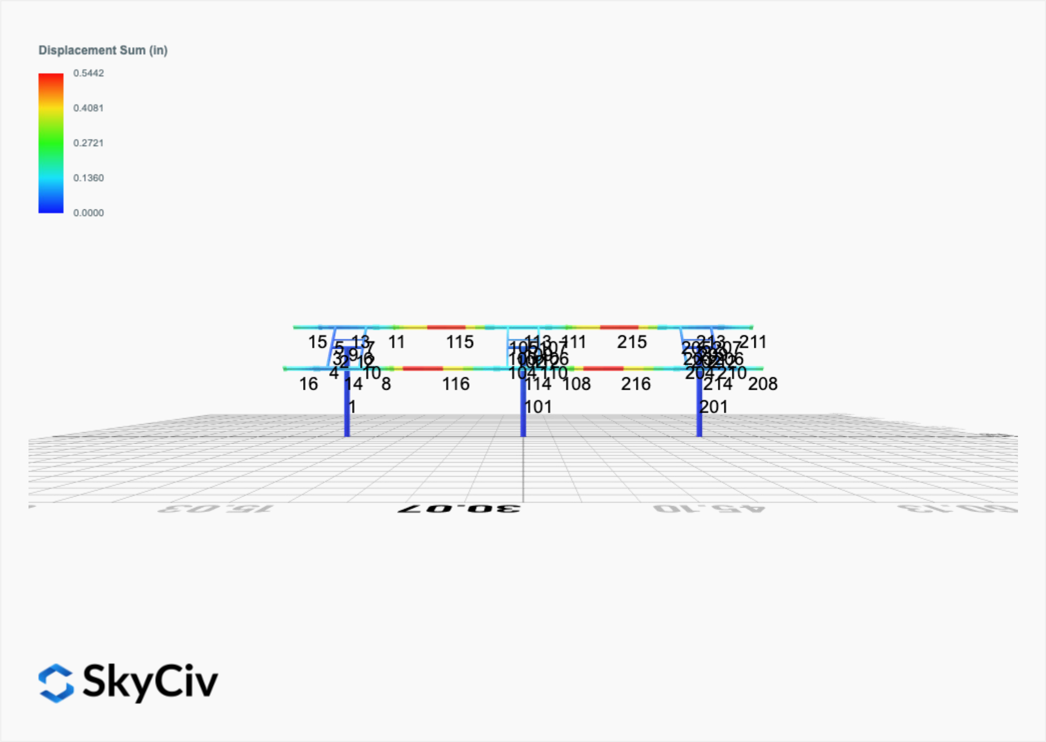
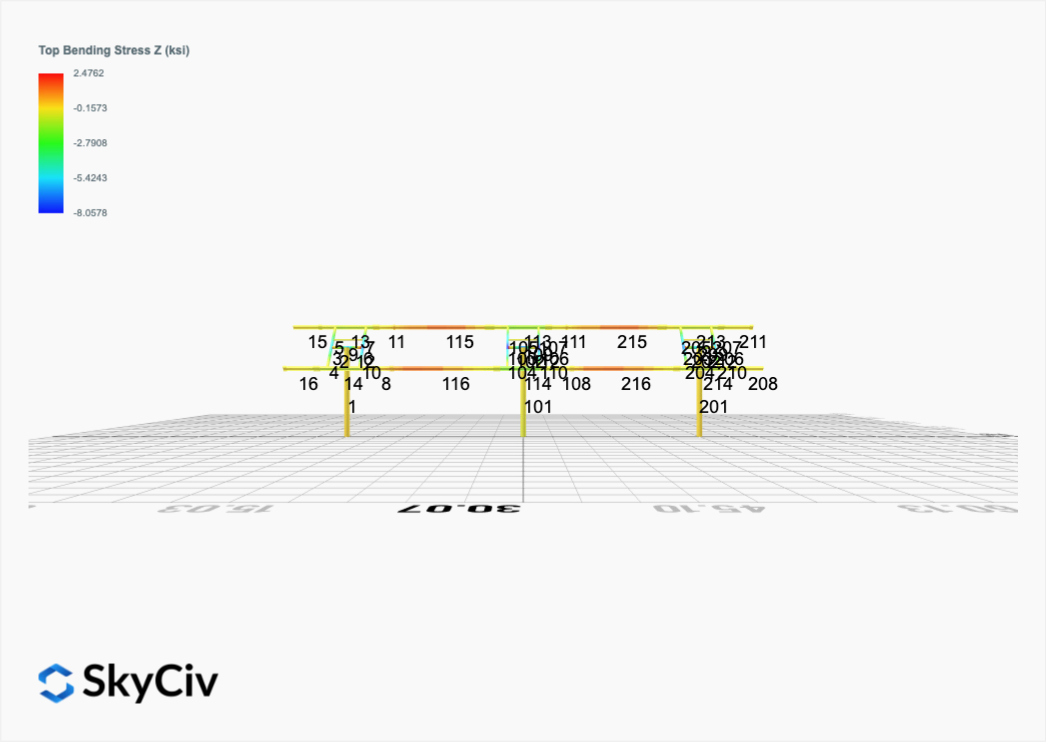
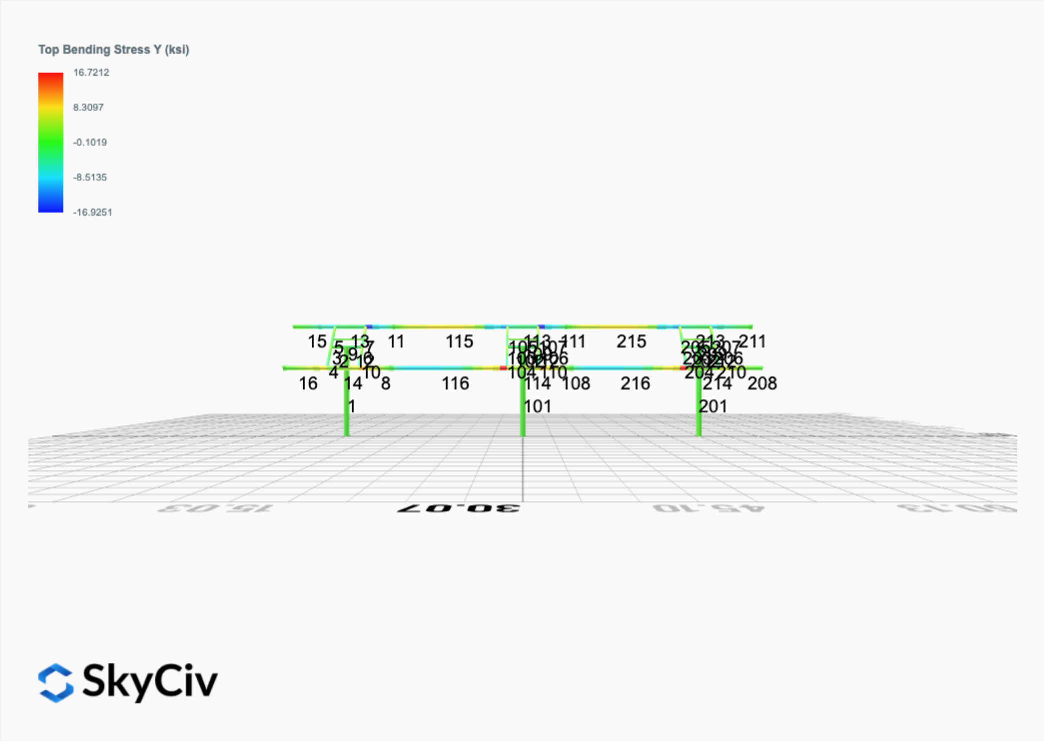
114
116
8
16
201
101
1
SkyCiv

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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8.3097
-0.1019
-8.5135
-16.9251
15
11
115
215
211
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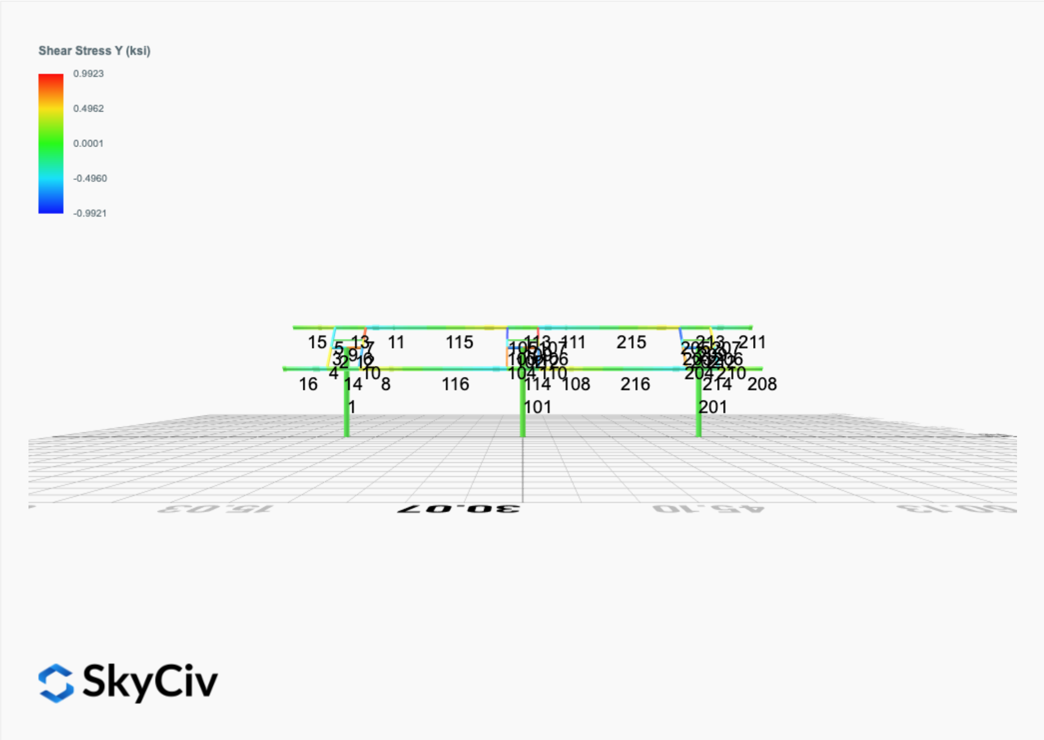
214
208
1
101
201


Shear Stress Y (ksi)
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0.4962
0.0001
-0.4960
-0.9921
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11
115
215
211
16
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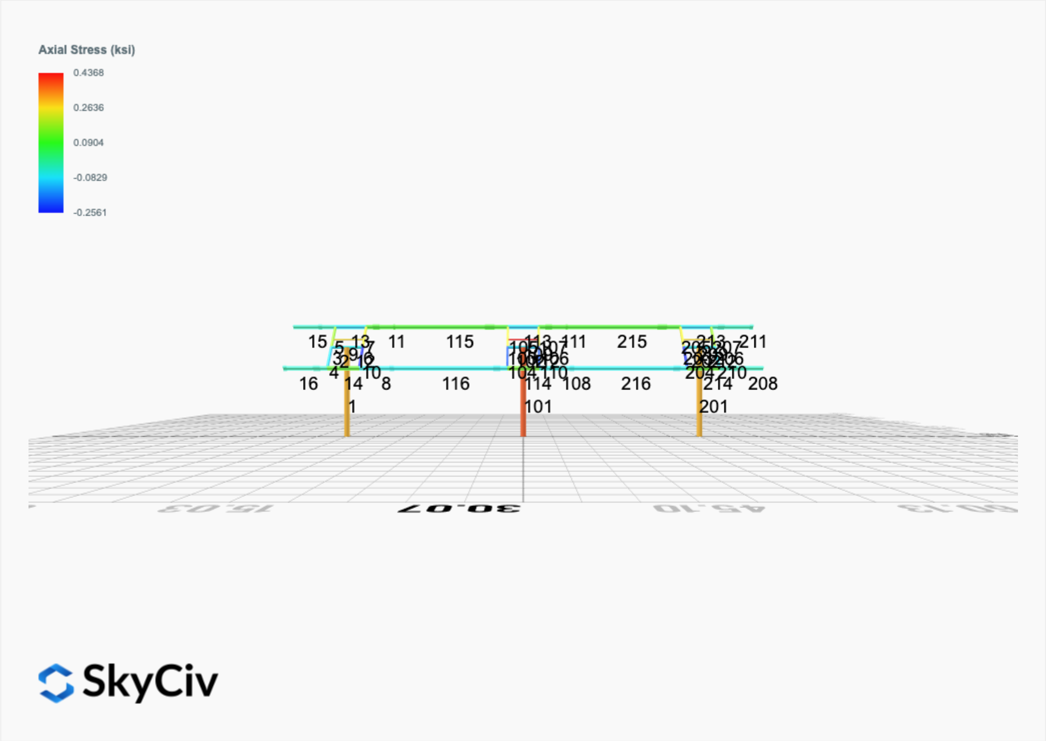
214
208
1
101
201


Axial Stress (ksi)
0.4368
0.2636
0.0904
-0.0829
-0.2561
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	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 2. D + L	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 3. D + (S or Lr or R)	0.0550	4.9273	0.1677	0.5731	-0.2274	-0.5468
ULS: 3. D + (S or Lr or R)	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0463	4.2389	0.1414	0.4830	-0.1917	-0.4584
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 5b. D + 0.7E	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0463	4.2389	0.1414	0.4830	-0.1917	-0.4584
ULS: 8. 0.6D + 0.7E	0.0123	1.3042	0.0374	0.1276	-0.0506	-0.1158
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7174	4.7928	0.2459	0.8005	-0.8786	31.6516
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7553	-0.4432	-0.1172	-0.3614	0.6938	-31.1725
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0071	6.2033	0.2790	0.9239	-0.7874	23.4251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0463	4.2389	0.1414	0.4830	-0.1917	-0.4584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0974	2.2763	0.0067	0.0525	0.3919	-23.6930
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0463	4.2389	0.1414	0.4830	-0.1917	-0.4584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0329	4.1380	0.2000	0.6535	-0.6801	23.6904
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0716	0.2110	-0.0723	-0.2179	0.4992	-23.4277
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0205	2.1737	0.0623	0.2126	-0.0844	-0.1931
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7256	3.9233	0.2209	0.7154	-0.8449	31.7288
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0123	1.3042	0.0374	0.1276	-0.0506	-0.1158
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7471	-1.3126	-0.1421	-0.4464	0.7275	-31.0953
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0123	1.3042	0.0374	0.1276	-0.0506	-0.1158

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	9.1981
Shear X	-4.6019
Shear Z	0.4363
Moment X	1.4259
Moment Y (Twist)	1.5074
Moment Z	53.4007

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	6.2033
Shear X	-2.7553
Shear Z	0.2790
Moment X	0.9239
Moment Y (Twist)	0.8786
Moment Z	31.7288

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	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 2. D + L	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 3. D + (S or Lr or R)	-0.1100	5.9932	-0.0000	-0.0011	-0.0002	1.1993
ULS: 3. D + (S or Lr or R)	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0927	5.1376	-0.0000	-0.0009	-0.0002	1.0131

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 5b. D + 0.7E	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0927	5.1376	-0.0000	-0.0009	-0.0002	1.0131
ULS: 8. 0.6D + 0.7E	-0.0246	1.5425	-0.0000	-0.0002	-0.0001	0.2727
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.4084	5.8723	-0.0000	-0.0012	-0.0011	39.1018
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.3327	-0.7352	0.0000	0.0004	0.0009	-37.0491
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6183	7.6137	-0.0000	-0.0015	-0.0010	29.9986
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0927	5.1376	-0.0000	-0.0009	-0.0002	1.0131
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4375	2.6581	-0.0000	-0.0004	0.0006	-27.1147
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0927	5.1376	-0.0000	-0.0009	-0.0002	1.0131
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.5666	5.0470	-0.0000	-0.0010	-0.0009	29.4400
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.4892	0.0913	0.0000	0.0002	0.0007	-27.6732
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0410	2.5708	-0.0000	-0.0004	-0.0001	0.4545
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.3920	4.8440	-0.0000	-0.0010	-0.0011	38.9200
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0246	1.5425	-0.0000	-0.0002	-0.0001	0.2727
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.3490	-1.7636	0.0000	0.0005	0.0010	-37.2309
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0246	1.5425	-0.0000	-0.0002	-0.0001	0.2727

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	11.3096
Shear X	-5.6911
Shear Z	-0.0000
Moment X	-0.0023
Moment Y (Twist)	0.0019
Moment Z	66.3012

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	7.6137
Shear X	-3.4084
Shear Z	-0.0000
Moment X	-0.0015
Moment Y (Twist)	0.0011
Moment Z	39.1018

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	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 2. D + L	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 3. D + (S or Lr or R)	0.0550	4.9272	-0.1677	-0.5753	0.2268	-0.5472
ULS: 3. D + (S or Lr or R)	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0464	4.2388	-0.1414	-0.4848	0.1912	-0.4587
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 5b. D + 0.7E	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0464	4.2388	-0.1414	-0.4848	0.1912	-0.4587
ULS: 8. 0.6D + 0.7E	0.0123	1.3042	-0.0374	-0.1280	0.0505	-0.1159
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7173	4.7926	-0.2458	-0.8028	0.8768	31.6500
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7552	-0.4431	0.1172	0.3621	-0.6923	-31.1714
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932

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	-2.0069	6.2031	-0.2790	-0.9269	0.7856	23.4237
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0464	4.2388	-0.1414	-0.4848	0.1912	-0.4587
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0974	2.2762	-0.0067	-0.0532	-0.3912	-23.6923
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0464	4.2388	-0.1414	-0.4848	0.1912	-0.4587
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0328	4.1379	-0.2000	-0.6554	0.6786	23.6892
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0715	0.2111	0.0723	0.2182	-0.4982	-23.4268
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0205	2.1736	-0.0623	-0.2134	0.0842	-0.1932
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7255	3.9232	-0.2209	-0.7174	0.8431	31.7273
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0123	1.3042	-0.0374	-0.1280	0.0505	-0.1159
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7470	-1.3126	0.1421	0.4474	-0.7260	-31.0941
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0123	1.3042	-0.0374	-0.1280	0.0505	-0.1159

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	9.1978
Shear X	-4.6017
Shear Z	-0.4363
Moment X	-1.4301
Moment Y (Twist)	1.5042
Moment Z	53.3987

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	6.2031
Shear X	-2.7552
Shear Z	-0.2790
Moment X	-0.9269
Moment Y (Twist)	0.8768
Moment Z	31.7273

Project Details

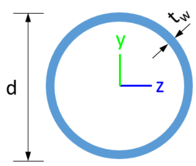
Design Code:	AISC 360-16 LRFD
Provision:	LRFD
Country:	United States
User Name:	sales@mtsolar.us
Project Name:	Vermillion Pivot 4x8-cu
Unit System:	imperial

Design Input Information

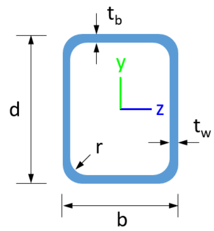
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

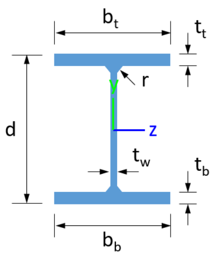
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
9	8in Pipe Sch 40	8.63	0.32					



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
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
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	9	23.95	23.95	11.41	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.19,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.17,1.18	300	200	1
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
8	19	1.33	1.33	2.05	1.68,1.68,1.68,1.68,1.68,1.68,1.66,1.68,1.63,1.68,1.65,1.68,1.64,1.68,1.67,1.68,1.79,1.68,1.66,1.68,1.63,1.68,1.65,1.68,1.65,1.68	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
11	19	1.33	1.33	2.05	1.77,1.77,1.77,1.77,1.77,1.77,1.88,1.77,1.99,1.77,1.89,1.77,1.94,1.77,1.83,1.77,1.52,1.77,1.87,1.77,2.02,1.77,1.89,1.77,1.92,1.77	300	200	1
12	5	1.30	1.30	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.08,1.09,1.09,1.08,1.09,1.09,1.07,1.09,1.06,1.09,1.07,1.09,1.07,1.09,1.08,1.08,1.13,1.08,1.07,1.09,1.07,1.09,1.07,1.09,1.07,1.09	300	200	1
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.10,1.09,1.08,1.09,1.07,1.09,1.08,1.09,1.08,1.09	300	200	1
15	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	9	23.95	23.95	11.41	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
105	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
107	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
108	19	1.33	1.33	2.05	2.26,2.26,2.26,2.26,2.26,2.26,2.30,2.26,2.35,2.26,2.31,2.26,2.33,2.26,2.28,2.26,1.99,2.26,2.30,2.26,2.35,2.26,2.31,2.26,2.32,2.26	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
111	19	1.33	1.33	2.05	2.34,2.34,2.34,2.35,2.34,2.34,2.08,2.34,1.62,2.34,2.06,2.34,1.77,2.34,2.11,2.35,1.51,2.35,2.09,2.34,1.56,2.34,2.03,2.34,1.83,2.34	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1

113	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.04,1.03,1.03,1.03,1.02,1.03,1.04,1.03,1.05,1.03,1.04,1.03,1.04,1.03	300	200	1
114	19	4.88	4.00	7.50	1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.02,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03,1.03	300	200	1
115	19	12.95	12.95	12.95	1.17,1.17,1.17,1.17,1.17,1.17,1.13,1.17,1.12,1.17,1.13,1.17,1.12,1.17,1.14,1.17,1.41,1.17,1.13,1.17,1.11,1.17,1.12,1.17,1.12,1.17	300	200	1
116	19	12.95	12.95	12.95	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.23,1.19,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
201	9	23.95	23.95	11.41	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.69,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
205	16	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
206	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.19,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.17,1.18	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
208	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.69,1.67,1.67,1.69,1.65,1.69,1.67,1.69,1.66,1.69	300	200	1
211	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.07,1.09,1.06,1.09,1.07,1.09,1.07,1.09,1.08,1.09,1.13,1.09,1.07,1.09,1.07,1.09,1.07,1.09,1.07,1.09	300	200	1
214	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.08,1.09,1.10,1.09,1.08,1.09,1.07,1.09,1.08,1.09,1.08,1.09	300	200	1
215	19	12.95	12.95	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.13,1.13,1.13,1.12,1.13,1.13,1.13,1.13,1.13,1.18,1.13,1.12,1.13,1.14,1.13,1.12,1.13,1.12,1.13	300	200	1
216	19	12.95	12.95	12.95	1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.15,1.14,1.13,1.14,1.13,1.14,1.13,1.14,1.13,1.14	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	187.68	83.29	83.29	113.39	113.39
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	24.37	6.12	40.24	43.62
14	133.20	85.85	24.61	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62

101	377.97	187.68	83.29	83.29	113.39	113.39
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	23.36	6.12	40.24	43.62
114	133.20	85.85	23.48	6.12	40.24	43.62
115	133.20	19.55	12.06	6.12	40.24	43.62
116	133.20	19.55	12.67	6.12	40.24	43.62
201	377.97	187.68	83.29	83.29	113.39	113.39
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	52.83	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	52.83	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	24.37	6.12	40.24	43.62
214	133.20	85.85	24.62	6.12	40.24	43.62
215	133.20	19.55	12.14	6.12	40.24	43.62
216	133.20	19.55	12.21	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	0.049	0.641	0.043	0.041	0.004	0.681	#13	0.489	Not Required	Pass
2	0.005	0.237	0.172	0.059	0.034	0.390	#13	0.035	Not Required	Pass
3	0.008	0.441	0.047	0.043	0.009	0.451	#13	0.045	Not Required	Pass
4	0.007	0.440	0.147	0.044	0.033	0.520	#13	0.080	Not Required	Pass
5	0.008	0.274	0.134	0.044	0.035	0.289	#13	0.074	Not Required	Pass
6	0.013	0.610	0.097	0.062	0.019	0.670	#13	0.045	Not Required	Pass
7	0.013	0.379	0.247	0.060	0.062	0.412	#13	0.074	Not Required	Pass
8	0.004	0.081	0.262	0.040	0.021	0.266	#23	0.095	Not Required	Pass
9	0.017	0.049	0.082	0.003	0.004	0.126	#13	0.204	Not Required	Pass
10	0.014	0.582	0.235	0.058	0.049	0.636	#21	0.080	Not Required	Pass
11	0.005	0.074	0.271	0.043	0.021	0.274	#23	0.095	Not Required	Pass
12	0.004	0.383	0.241	0.089	0.046	0.619	#13	0.053	Not Required	Pass
13	0.007	0.173	0.570	0.053	0.026	0.691	#21	0.286	Not Required	Pass
14	0.009	0.164	0.558	0.051	0.026	0.657	#21	0.190	Not Required	Pass
15	0.000	0.050	0.142	0.022	0.011	0.186	#21	Not Required	Not Required	Pass

15	0.000	0.050	0.142	0.022	0.011	0.186	#21	Not Required	Not Required	Pass
16	0.000	0.050	0.142	0.022	0.011	0.186	#21	Not Required	Not Required	Pass
101	0.060	0.796	0.000	0.050	0.000	0.824	#13	0.489	Not Required	Pass
102	0.006	0.386	0.262	0.092	0.048	0.634	#13	0.035	Not Required	Pass
103	0.012	0.640	0.062	0.064	0.003	0.675	#13	0.045	Not Required	Pass
104	0.012	0.660	0.243	0.066	0.050	0.748	#13	0.080	Not Required	Pass
105	0.012	0.397	0.256	0.063	0.066	0.437	#13	0.074	Not Required	Pass
106	0.012	0.640	0.062	0.064	0.003	0.675	#13	0.045	Not Required	Pass
107	0.012	0.397	0.256	0.063	0.066	0.437	#13	0.074	Not Required	Pass
108	0.004	0.062	0.272	0.047	0.022	0.329	#21	0.095	Not Required	Pass
109	0.026	0.043	0.055	0.001	0.000	0.106	#13	0.204	Not Required	Pass
110	0.012	0.660	0.243	0.066	0.050	0.748	#13	0.080	Not Required	Pass
111	0.005	0.067	0.279	0.044	0.022	0.317	#21	0.095	Not Required	Pass
112	0.006	0.386	0.262	0.092	0.048	0.634	#13	0.035	Not Required	Pass
113	0.007	0.204	0.580	0.054	0.026	0.763	#21	0.286	Not Required	Pass
114	0.011	0.260	0.571	0.057	0.027	0.787	#21	0.286	Not Required	Pass
115	0.032	0.460	0.296	0.044	0.021	0.702	#21	0.925	Not Required	Pass
116	0.012	0.424	0.294	0.047	0.022	0.662	#21	0.925	Not Required	Pass
201	0.049	0.641	0.042	0.041	0.004	0.681	#13	0.489	Not Required	Pass
202	0.004	0.383	0.241	0.089	0.046	0.619	#13	0.053	Not Required	Pass
203	0.013	0.610	0.097	0.062	0.019	0.670	#13	0.045	Not Required	Pass
204	0.014	0.582	0.235	0.058	0.049	0.636	#21	0.080	Not Required	Pass
205	0.013	0.379	0.247	0.060	0.062	0.412	#13	0.074	Not Required	Pass
206	0.008	0.441	0.047	0.043	0.009	0.452	#13	0.045	Not Required	Pass
207	0.008	0.274	0.134	0.044	0.035	0.290	#13	0.074	Not Required	Pass
208	0.000	0.050	0.142	0.022	0.011	0.186	#21	Not Required	Not Required	Pass
209	0.017	0.049	0.082	0.003	0.004	0.126	#13	0.204	Not Required	Pass
210	0.007	0.440	0.147	0.044	0.033	0.520	#13	0.080	Not Required	Pass
211	0.000	0.050	0.142	0.022	0.011	0.186	#21	Not Required	Not Required	Pass
212	0.005	0.237	0.172	0.059	0.034	0.390	#13	0.035	Not Required	Pass
213	0.007	0.173	0.570	0.053	0.026	0.691	#21	0.190	Not Required	Pass
214	0.009	0.164	0.558	0.051	0.026	0.657	#21	0.286	Not Required	Pass
215	0.032	0.463	0.296	0.043	0.021	0.709	#21	0.925	Not Required	Pass
216	0.012	0.443	0.294	0.040	0.021	0.679	#21	0.925	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS

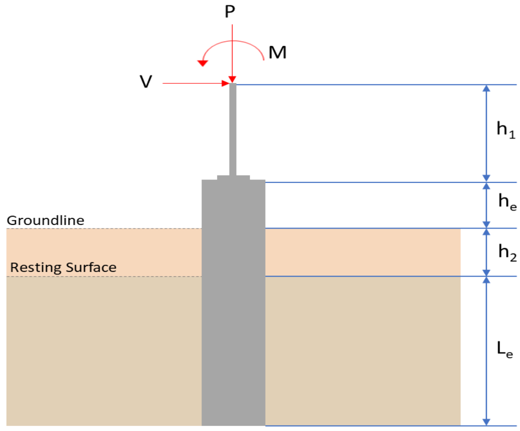
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 6.75\ \mathrm{ft}$ - Total pile length
$h_1 = 0\ \mathrm{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \mathrm{ft}$ - Depth to resisting surface
$h_e = 0\ \mathrm{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.203	9.198
V_x (kip)	-2.755	-4.602
V_z (kip)	0.279	0.436
M_x (kipft)	0.924	1.426
M_z (kipft)	31.729	53.401

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.755\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.43869\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.729\ \text{kipft}) + ((-2.755\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 5.0524\ \text{kipft/ft}$$

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:
$L_{e,x} = 6.2188\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.279\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.044427\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.924\ \text{kipft}) + ((0.279\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.14713\ \text{kipft/ft}$$

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:
$L_{e,z} = 2.6622\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2188\ \text{ft}), (2.6622\ \text{ft})]$$

$$L_{e,req} = 6.219\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.219\ \text{ft})}{(6.75\ \text{ft})}$$

$$Ratio = 0.92133$$

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

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

$$q = \frac{(6.203\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.38769\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.38769\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.19384$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.6875$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.43869\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.0524\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0524\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.43869\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (5.0524\ \text{kipft/ft})) + (4 \times (-0.43869\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.658\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.0524\ \text{kipft/ft})) + (3 \times (-0.43869\ \text{kip/ft}) \times (6.75\ \text{ft}))]^{\ 2}}{(6.75\ \text{ft})^2 \times [(3 \times (5.0524\ \text{kipft/ft})) + (2 \times (-0.43869\ \text{kip/ft}) \times (6.75\ \text{ft}))]}$$

$$p = 0.22865\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (5.0524\ \text{kipft/ft})) + ((-0.43869\ \text{kip/ft}) \times (6.75\ \text{ft}))]}{(6.75\ \text{ft})^2}$$

$$s = 0.94072\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.658\ \text{ft})}{2}$$

$$p_a = 0.34935\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22865\ \mathrm{kip/ft^2}\right)}{\left(0.34935\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65451$$

Status: **PASS**
Ratio: **0.650**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.75\ \mathrm{ft})$$

$$p_s = 1.0125\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.94072\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.92911$$

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

Considering z-direction:

$H_o = 0.044427\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.14713\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14713\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.14713\ \mathrm{kipft/ft})) + (4 \times (0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.824\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.14713\ \mathrm{kipft/ft})) + (3 \times (0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^2}{(6.75\ \mathrm{ft})^2 \times [(3 \times (0.14713\ \mathrm{kipft/ft})) + (2 \times (0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = 0.035014\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.14713\ \mathrm{kipft/ft})) + ((0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})]}{(6.75\ \mathrm{ft})^2}$$

$$s = 0.078242\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.824\ \mathrm{ft})}{2}$$

$$p_a = 0.3618\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.035014\ \mathrm{kip/ft^2}\right)}{\left(0.3618\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.096777$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.75\ \mathrm{ft})$$

$$p_s = 1.0125\ \mathrm{kip/ft^2}$$

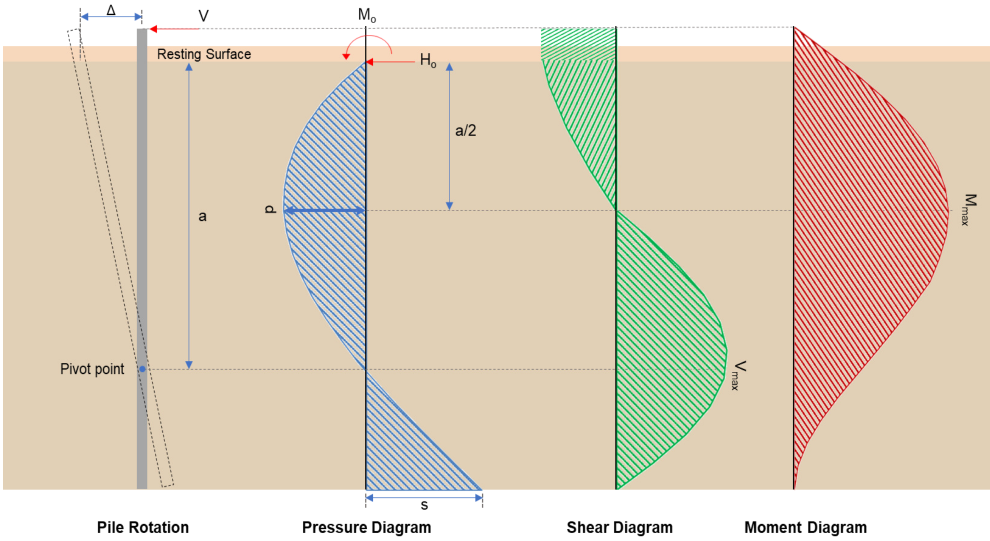
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.078242\ \mathrm{kip/ft^2})}{(1.0125\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.077276$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.602\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.7328\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.401\ \mathrm{kipft}) + ((-4.602\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.5033\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.5033\ \mathrm{kipft/ft})}{(-0.7328\ \mathrm{kip/ft})}$$

$$E = 11.604\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.5033\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.7328\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.5033 \text{ kipft/ft})) + (4 \times (-0.7328 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6572 \text{ ft}$$

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

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

$$V_{max} = ((-0.7328 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.604 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.6572 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.604 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.6572 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 10.85 \text{ kip}$$

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

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

$$M_{max} = ((-0.7328 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.604 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6572 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (11.604 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.6572 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.604 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.6572 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 34.822 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.436 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.069427 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.426 \text{ kipft}) + ((0.436 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.22707 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.22707 \text{ kipft/ft})}{(0.069427 \text{ kip/ft})}$$

$$E = 3.2706 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.22707 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (0.069427 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.22707 \text{ kipft/ft})) + (4 \times (0.069427 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8257 \text{ ft}$$

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

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

$$V_{max} = ((0.069427 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2706 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8257 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.2706 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8257 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.42322 \text{ kip}$$

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

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

$$M_{max} = ((0.069427 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.2706 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8257 \text{ ft})}{2 \times (6.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (3.2706 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.8257 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.2706 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.8257 \text{ ft})}{2 \times (6.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 1.2612 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.198 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.29 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.29 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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,

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

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

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

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,

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

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

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.198\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0034383$$

Status: PASS
Ratio: 0.000

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\ D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.198 \text{ kip} \rightarrow 9198 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9198 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.71 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.89 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.85 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(10.85\ \text{kip})}{(110.89\ \text{kip})}$$ $$Ratio = 0.09784$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $V_{max} = 0.42322\ \text{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.42322\ \text{kip})}{(110.89\ \text{kip})}$$ $$Ratio = 0.0038165$$	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:** $M_{max} = 34.822\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(34.822\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.13951$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 1.2612\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(1.2612\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.0050531$$	Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

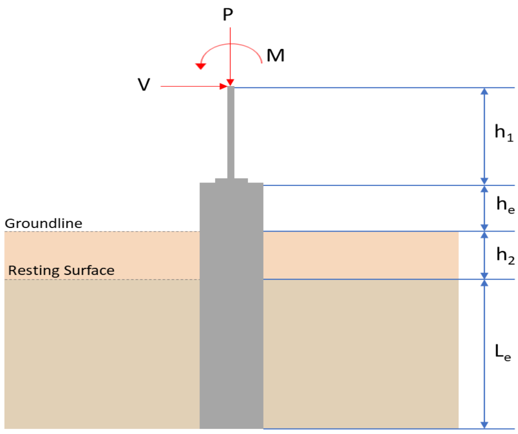
SkyCiv Foundation Design

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
$L = 6.75$ 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)	6.203	9.198
V_x (kip)	-2.755	-4.602
V_z (kip)	-0.279	-0.436
M_x (kipft)	-0.927	-1.430
M_z (kipft)	31.727	53.399

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,

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

$$H_o = \frac{(-2.755\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.43869\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.727\ \mathrm{kipft}) + ((-2.755\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.0521\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 6.2186\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.279\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.044427\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.927\ \mathrm{kipft}) + ((-0.279\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.14761\ \mathrm{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.8914\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.2186\ \mathrm{ft}), (1.8914\ \mathrm{ft})]$$

$$L_{e,req} = 6.219\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 6.75\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.219\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.92133$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

	$$q = \frac{P_v}{A}$$ $$q = \frac{(6.203\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$ $$q = 0.38769\ \mathrm{kip/ft^2}$$ **Check bearing capacity ratio:** $Ratio$ - Capacity $$Ratio = \frac{q}{q_a}$$ $$Ratio = \frac{(0.38769\ \mathrm{kip/ft^2})}{(2000\ \mathrm{psf})}$$ $$Ratio = 0.19384$$	Status: **PASS** Ratio: **0.190**
Czerniak	**Lateral Soil Pressure (ASD):** L/D - Length to least lateral dimension ratio, $$L/D = \frac{L}{D}$$ $$L/D = \frac{(6.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$ $$L/D = 1.6875$$ Since L/D $\leq$ 10, Pile is short. **Considering x-direction:** $H_o = -0.43869\ \mathrm{kip/ft}$ - Lateral force per length of pile, $M_o = 5.0521\ \mathrm{kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point, $$a = \frac{(4\,M_o\,L_e) + (3\,H_o\,L_e^2)}{(6\,M_o) + (4\,H_o\,L_e)}$$ $$a = \frac{(4 \times (5.0521\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.43869\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (5.0521\ \mathrm{kipft/ft})) + (4 \times (-0.43869\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$ $$a = 4.658\ \mathrm{ft}$$ p - Earth pressure against the pile at distance a/2 from resting surface, $$p = \frac{0.75\,[(4\,M_o) + (3\,H_o\,L_e)]^2}{L_e^2\,[(3\,M_o) + (2\,H_o\,L_e)]}$$ $$p = \frac{0.75 \times [(4 \times (5.0521\ \mathrm{kipft/ft})) + (3 \times (-0.43869\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^2}{(6.75\ \mathrm{ft})^2 \times [(3 \times (5.0521\ \mathrm{kipft/ft})) + (2 \times (-0.43869\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$ $$p = 0.22863\ \mathrm{kip/ft^2}$$ s - Earth pressure against the pile at distance L_e, $$s = \frac{6\,[(2\,M_o) + (H_o\,L_e)]}{L_e^2}$$ $$s = \frac{6 \times [(2 \times (5.0521\ \mathrm{kipft/ft})) + ((-0.43869\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$ $$s = 0.94064\ \mathrm{kip/ft^2}$$ **Check lateral soil pressure capacity:** p_a - Allowable lateral soil pressure at depth a/2, $$p_a = R\,\frac{a}{2}$$ $$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.658\ \mathrm{ft})}{2}$$ $$p_a = 0.34935\ \mathrm{kip/ft^2}$$ $Ratio$ - Lateral soil capacity	

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

$$Ratio = \frac{\left(0.22863\ \mathrm{kip/ft^2}\right)}{\left(0.34935\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.65443$$

Status: **PASS**
Ratio: **0.650**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.75\ \mathrm{ft})$$

$$p_s = 1.0125\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.94064\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.92902$$

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

Considering z-direction:

$H_o = -0.044427\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.14761\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14761\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.14761\ \mathrm{kipft/ft})) + (4 \times (-0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8236\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.14761\ \mathrm{kipft/ft})) + (3 \times (-0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]^2}{(6.75\ \mathrm{ft})^2 \times [(3 \times (0.14761\ \mathrm{kipft/ft})) + (2 \times (-0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}$$

$$p = -0.010028\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.14761\ \mathrm{kipft/ft})) + ((-0.044427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))]}{(6.75\ \mathrm{ft})^2}$$

$$s = -0.00061335\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.8236\ \mathrm{ft})}{2}$$

$$p_a = 0.36177\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.010028\ \mathrm{kip/ft^2}\right)}{\left(0.36177\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.02772$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.75\ \mathrm{ft})$$

$$p_s = 1.0125\ \mathrm{kip/ft^2}$$

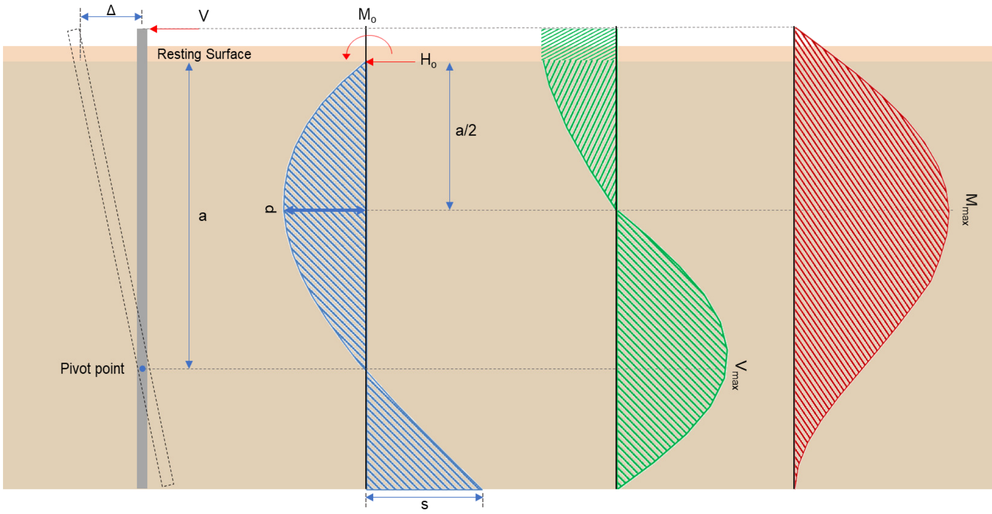
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.00061335\ \mathrm{kip/ft^2}\right)}{\left(1.0125\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.00060578$$

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



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.602\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.7328\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(53.399\ \mathrm{kipft}) + ((-4.602\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 8.503\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.503\ \mathrm{kipft/ft})}{(-0.7328\ \mathrm{kip/ft})}$$

$$E = 11.603\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.503\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.7328\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (8.503\ \mathrm{kipft/ft})) + (4 \times (-0.7328\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.6572\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.7328\ \mathrm{kip/ft}) \times (48\ \mathrm{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.603\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6572\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.603\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6572\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.85\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.7328\ \mathrm{kip/ft}) \times (48\ \mathrm{in}) \times (6.75\ \mathrm{ft})) \times \left[\left(\frac{(11.603\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + \frac{(4.6572\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4 \times (11.603\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 3\right) \times \left(\frac{(4.6572\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.603\ \mathrm{ft})}{(6.75\ \mathrm{ft})} + 2\right) \times \left(\frac{(4.6572\ \mathrm{ft})}{2 \times (6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 34.821\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.436\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.069427\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.43\ \mathrm{kipft}) + ((-0.436\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.22771\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.22771\ \mathrm{kipft/ft})}{(-0.069427\ \mathrm{kip/ft})}$$

$$E = 3.2798\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.22771\ \mathrm{kipft/ft}) \times (6.75\ \mathrm{ft})) + \left(3 \times (-0.069427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft})^2\right)}{(6 \times (0.22771\ \mathrm{kipft/ft})) + (4 \times (-0.069427\ \mathrm{kip/ft}) \times (6.75\ \mathrm{ft}))}$$

$$a = 4.8254\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.069427\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2798\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.8254\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (3.2798\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.8254\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.42388\ \text{kip}$$

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

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

$$M_{max} = ((-0.069427\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(3.2798\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.8254\ \text{ft})}{2 \times (6.75\ \text{ft})}\right) - \left[\left(\frac{4 \times (3.2798\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.8254\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (3.2798\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.8254\ \text{ft})}{2 \times (6.75\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 1.2635\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.198\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.29\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.29\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

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, $$s_{rebar} = Max\,[1.5, (1.5\, d_{bar})]$$ $$s_{rebar} = Max\,[1.5, (1.5 \times (0.625\text{ in}))]$$ $$s_{rebar} = 1.5\text{ in}$$ **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, $$s_{ties} = Min\,[(16\, d_{bar}), (48\, d_{ties}), Min\,(D, b)]$$ $$s_{ties} = Min\,[(16 \times (0.625\text{ in})), (48 \times (0.375\text{ in})), Min\,((48\text{ in}), (48\text{ in}))]$$ $$s_{ties} = 10\text{ in}$$ **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 $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.198\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0034383$$	Status: PASS Ratio: 0.000
22.5.2.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48\text{ in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\text{ in})$$ $$d = 38.4\text{ in}$$	
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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.198 \text{ kip} \rightarrow 9198 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9198 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.71 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.89 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.85 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(10.85\ \mathrm{kip})}{(110.89\ \mathrm{kip})}$$ $$Ratio = 0.097837$$	Status: **PASS** Ratio: **0.100**
	Considering z-direction: $V_{max} = 0.42388\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.42388\ \mathrm{kip})}{(110.89\ \mathrm{kip})}$$ $$Ratio = 0.0038224$$	Status: **PASS** Ratio: **0.000**
14.5.2.1b	**Flexural Strength (ACI 318-19, LRFD)** S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$ $\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5\ \mathrm{ksi}) \times \left(18432\ \mathrm{in}^3\right)$$ $$\phi M_{n,2} = 2121.6\ \mathrm{kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6\ \mathrm{kipft}), (2121.6\ \mathrm{kipft})]$$ $$\phi M_n = 249.6\ \mathrm{kipft}$$ **Considering x-direction:** $M_{max} = 34.821\ \mathrm{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(34.821\ \mathrm{kipft})}{(249.6\ \mathrm{kipft})}$$ $$Ratio = 0.13951$$	Status: **PASS** Ratio: **0.140**
	Considering z-direction: $M_{max} = 1.2635\ \mathrm{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$	

	$$Ratio = \frac{(1.2635\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.005062$$		Status: **PASS** Ratio: **0.010**

REFERENCES	CALCULATIONS	RESULTS

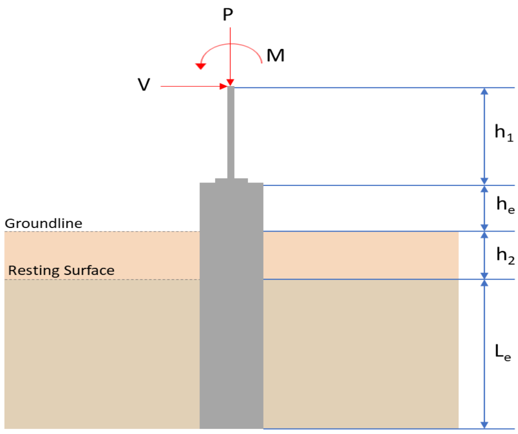
SkyCiv Foundation Design

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
$L = 7.25$ 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)	7.614	11.310
V_x (kip)	-3.408	-5.691
V_z (kip)	0.000	0.000
M_x (kipft)	-0.002	-0.002
M_z (kipft)	39.102	66.301

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,

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

$$H_o = \frac{(-3.408\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.54268\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.102\ \text{kipft}) + ((-3.408\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 6.2264\ \text{kipft/ft}$$

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:
$L_{e,x} = 6.5737\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.002\ \text{kipft}) + ((0\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.00031847\ \text{kipft/ft}$$

L_e - Required depth of embedment in earth,

$$L_e = 2.29\, \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.00031847\ \text{kipft/ft})}{(150\ \text{psf/ft})}}$$

$$L_e = 0.29433\ \text{ft}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.5737\ \text{ft}), (0.29433\ \text{ft})]$$

$$L_{e,req} = 6.574\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 7.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.574\ \text{ft})}{(7.25\ \text{ft})}$$

$$Ratio = 0.90676$$

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

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

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

$$q = \frac{(7.614 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47588 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.47588 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.23794$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.8125$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.54268 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 6.2264 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.2264 \text{ kipft/ft}) \times (7.25 \text{ ft})) + \left(3 \times (-0.54268 \text{ kip/ft}) \times (7.25 \text{ ft})^2\right)}{(6 \times (6.2264 \text{ kipft/ft})) + (4 \times (-0.54268 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0124 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.2264 \text{ kipft/ft})) + (3 \times (-0.54268 \text{ kip/ft}) \times (7.25 \text{ ft}))]^{\ 2}}{(7.25 \text{ ft})^2 \times [(3 \times (6.2264 \text{ kipft/ft})) + (2 \times (-0.54268 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.2266 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (6.2264 \text{ kipft/ft})) + ((-0.54268 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.97238 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0124 \text{ ft})}{2}$$

$$p_a = 0.37593 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.2266 \text{ kip/ft}^2\right)}{\left(0.37593 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.60276$$

Status: **PASS**
Ratio: **0.600**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.97238 \text{ kip/ft}^2\right)}{\left(1.0875 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.89414$$

Status: **PASS**
Ratio: **0.890**

Considering z-direction:

$H_o = 0 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.00031847 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (7.25 \text{ ft})) + \left(3 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft})^2\right)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 4.8333 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.00031847 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft}))]^{\ 2}}{(7.25 \text{ ft})^2 \times [(3 \times (0.00031847 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.000024236 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.00031847 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 0.000072707 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8333 \text{ ft})}{2}$$

$$p_a = 0.3625 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.000024236\ \mathrm{kip/ft^2}\right)}{\left(0.3625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.000066857$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = \left(150\ \mathrm{psf/ft}\right) \times \left(7.25\ \mathrm{ft}\right)$$

$$p_s = 1.0875\ \mathrm{kip/ft^2}$$

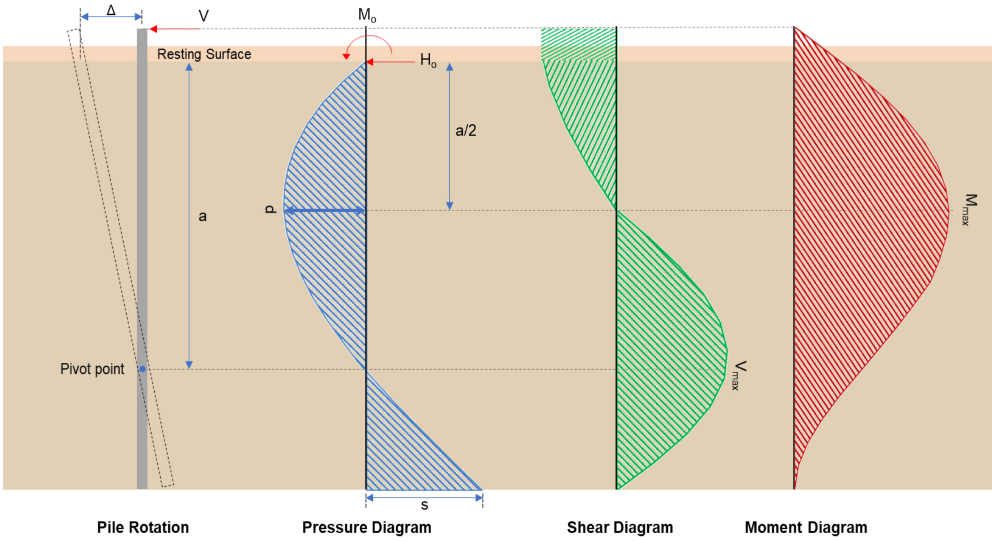
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.000072707\ \mathrm{kip/ft^2}\right)}{\left(1.0875\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.000066857$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{\left(-5.691\ \mathrm{kip}\right)}{1.57 \times \left(48\ \mathrm{in}\right)}$$

$$H_o = -0.90621\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{\left(66.301\ \mathrm{kipft}\right) + \left(\left(-5.691\ \mathrm{kip}\right) \times \left(0\ \mathrm{ft}\right)\right)}{1.57 \times \left(48\ \mathrm{in}\right)}$$

$$M_o = 10.557\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{\left(10.557\ \mathrm{kipft/ft}\right)}{\left(-0.90621\ \mathrm{kip/ft}\right)}$$

$$E = 11.65\ \mathrm{ft}$$

a - Distance from resisting surface to pivot point,

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

$$(6\,M_o) + (4\,H_o\,L_e)$$

$$a = \frac{(4 \times (10.557\ \text{kipft/ft}) \times (7.25\ \text{ft})) + \left(3 \times (-0.90621\ \text{kip/ft}) \times (7.25\ \text{ft})^2\right)}{(6 \times (10.557\ \text{kipft/ft})) + (4 \times (-0.90621\ \text{kip/ft}) \times (7.25\ \text{ft}))}$$

$$a = 5.0105\ \text{ft}$$

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

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

$$V_{max} = ((-0.90621\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.65\ \text{ft})}{(7.25\ \text{ft})} + 3\right) \times \left(\frac{(5.0105\ \text{ft})}{(7.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (11.65\ \text{ft})}{(7.25\ \text{ft})} + 2\right) \times \left(\frac{(5.0105\ \text{ft})}{(7.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 12.697\ \text{kip}$$

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

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

$$M_{max} = ((-0.90621\ \text{kip/ft}) \times (48\ \text{in}) \times (7.25\ \text{ft})) \times \left[\left(\frac{(11.65\ \text{ft})}{(7.25\ \text{ft})} + \frac{(5.0105\ \text{ft})}{2 \times (7.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (11.65\ \text{ft})}{(7.25\ \text{ft})} + 3\right) \times \left(\frac{(5.0105\ \text{ft})}{2 \times (7.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (11.65\ \text{ft})}{(7.25\ \text{ft})} + 2\right) \times \left(\frac{(5.0105\ \text{ft})}{2 \times (7.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 43.644\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.002\ \text{kipft}) + ((0\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.00031847\ \text{kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00031847\ \text{kipft/ft}) \times (7.25\ \text{ft})) + \left(3 \times (0\ \text{kip/ft}) \times (7.25\ \text{ft})^2\right)}{(6 \times (0.00031847\ \text{kipft/ft})) + (4 \times (0\ \text{kip/ft}) \times (7.25\ \text{ft}))}$$

$$a = 4.8333\ \text{ft}$$

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

$$V_{max} = 12\left(\frac{M_o\,b}{L_e}\right)\left(\frac{a}{L_e} - 1\right)\left(\frac{a}{L_e}\right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00031847\ \text{kipft/ft}) \times (48\ \text{in})}{(7.25\ \text{ft})}\right) \times \left(\frac{(4.8333\ \text{ft})}{(7.25\ \text{ft})} - 1\right) \times \left(\frac{(4.8333\ \text{ft})}{(7.25\ \text{ft})}\right)^2$$

$$V_{max} = 0.00031237\ \text{kip}$$

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

$$M_{max} = (M_o\, \delta) \left[1 - \left(4\, \frac{\ }{2\, L_e}\right) + \left(3\, \frac{\ }{2\, L_e}\right)\right]$$

$$M_{max} = ((0.00031847 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.8333 \text{ ft})}{2 \times (7.25 \text{ ft})}^{3}\right) + \left(3 \times \frac{(4.8333 \text{ ft})}{2 \times (7.25 \text{ ft})}^{4}\right)\right]$$

$$M_{max} = 0.0011323 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.31 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.22 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.22 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

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**	
22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, [A_g - A_{st}]\right) + (f_{yk}\, A_{st})\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5 \text{ ksi}) \times \left[\left(2304 \text{ in}^2\right) - \left(4.2951 \text{ in}^2\right)\right]\right) + \left((60 \text{ ksi}) \times \left(4.2951 \text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2 \text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(11.31 \text{ kip})}{(2675.2 \text{ kip})}$$ $$Ratio = 0.0042277$$	Status: **PASS** Ratio: **0.000**
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	**Shear Strength (ACI 318-19, LRFD)** **Parameters:** $b_w = 48 \text{ in}$ - 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.31 \text{ kip} \rightarrow 11310 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11310 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$ $$V_{c,a} = 120 \text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$	

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (120\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 120\ \mathrm{kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$

$$\phi V_n = 111.08\ \mathrm{kip}$$

Considering x-direction:

$V_{max} = 12.697\ \mathrm{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(12.697\ \mathrm{kip})}{(111.08\ \mathrm{kip})}$$

$$Ratio = 0.11431$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

	$$S_m = 18432 \text{ in}^3$$ $\lambda = 1$ - 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:** $M_{max} = 43.644 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(43.644 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.17486$$	Status: **PASS** Ratio: **0.170**
	Considering z-direction: $M_{max} = 0.0011323 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.0011323 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 4.5366\times10^{-6}$$	Status: **PASS** Ratio: **0.000**