Your Project Calculations

Project Name: Tavoli Apartments-cuV1

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Tavoli%20Apartments-cuV1&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=YZLvVXQCwCbjv6gnMQHnBXJoPicCL71GNKdMGhX8IeqMh41QjvcJt0IkzfZoRZmI

Array Specification

Product:	Beam
Unique ID:	3P-22.5-8TOP-HD-45-L-4Hx8W-AD77
Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Desired Tilt Angle:	10
Front Edge Clearance:	14
Total Array Height at Tilt:	16.60 ft
Total Frame Length:	60.00 ft
Frame Weight:	3156 lbs
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	60.46 ft
Rail Length:	180.60 in
Rail Spacing:	3.74 ft
Rail Check:	Not Checked

Support Specifications

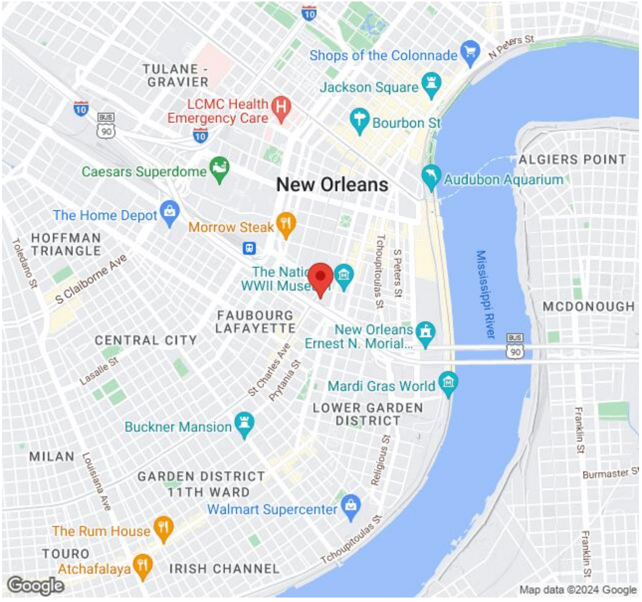
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	15.31 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.00 ft Pile 3: 5.75 ft
Foundation Volume:	10.370 y^3
Foundation Result:	PASSED
Mount Twist:	0.239348 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	1040 St Charles Ave, New Orleans, LA 70130, USA
Wind Speed:	134 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

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

AutoDesigner Input

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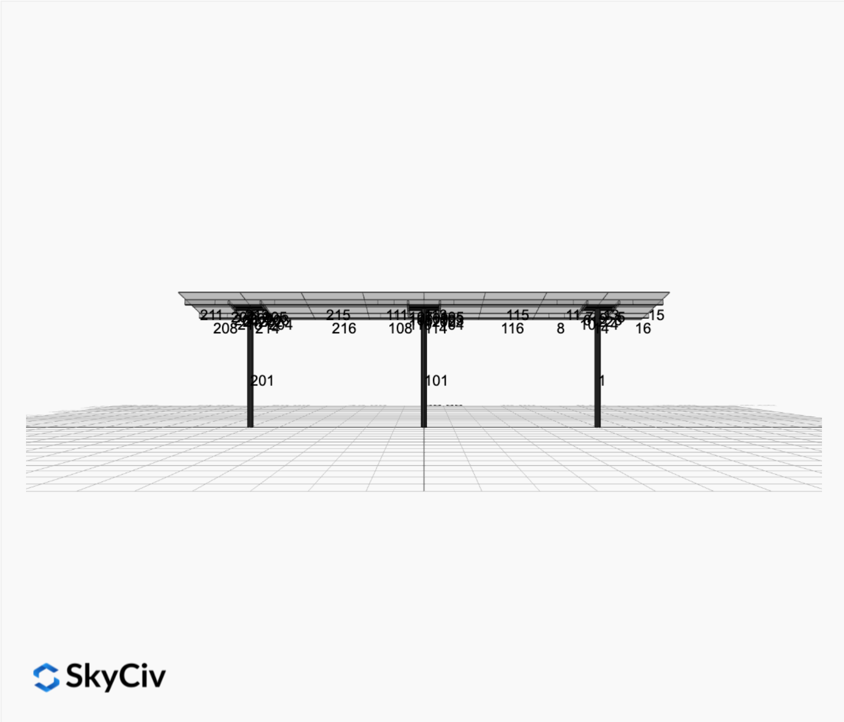
Design Notes:

- 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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201
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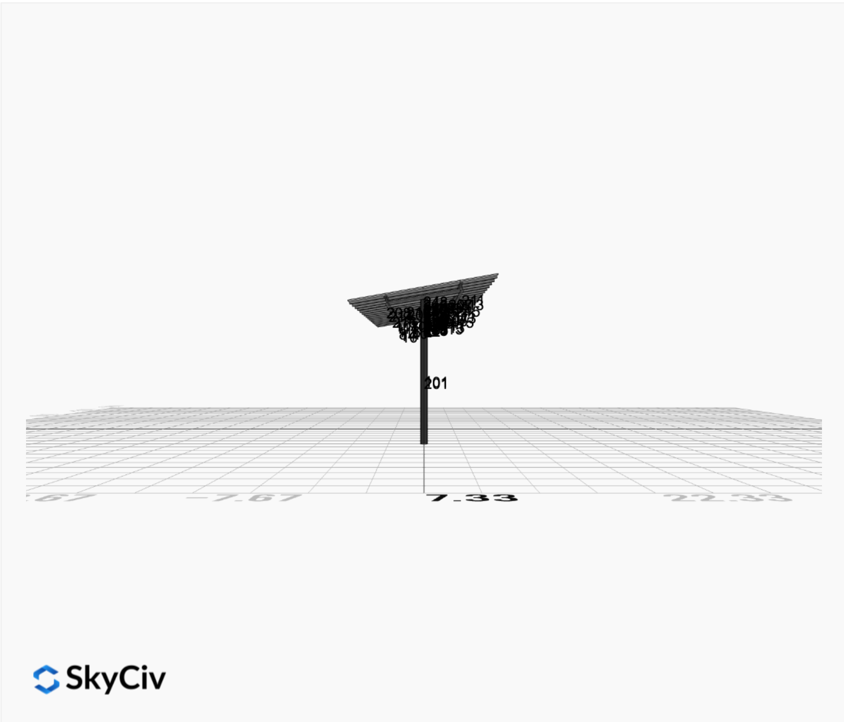
SkyCiv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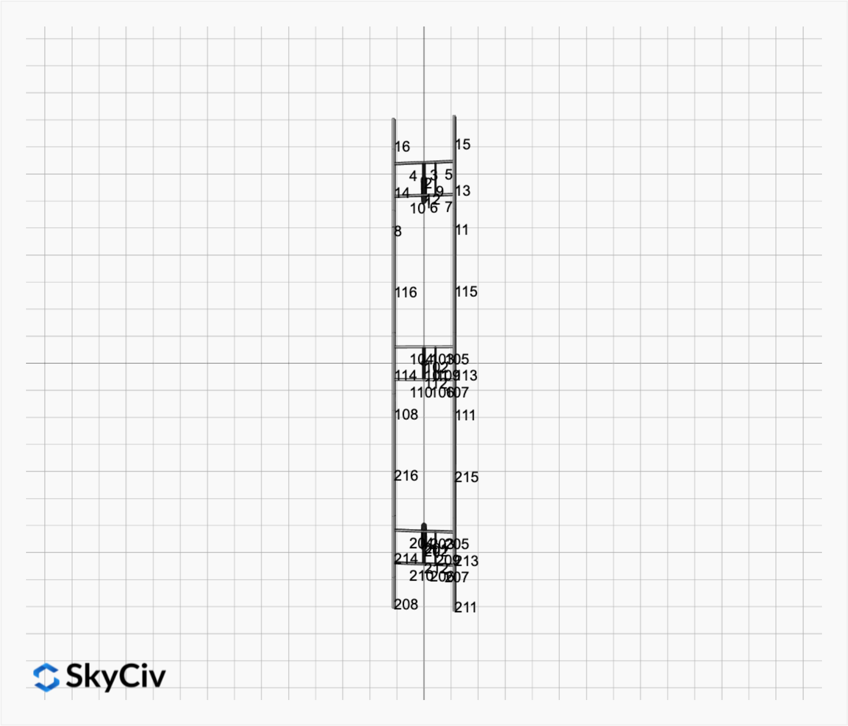
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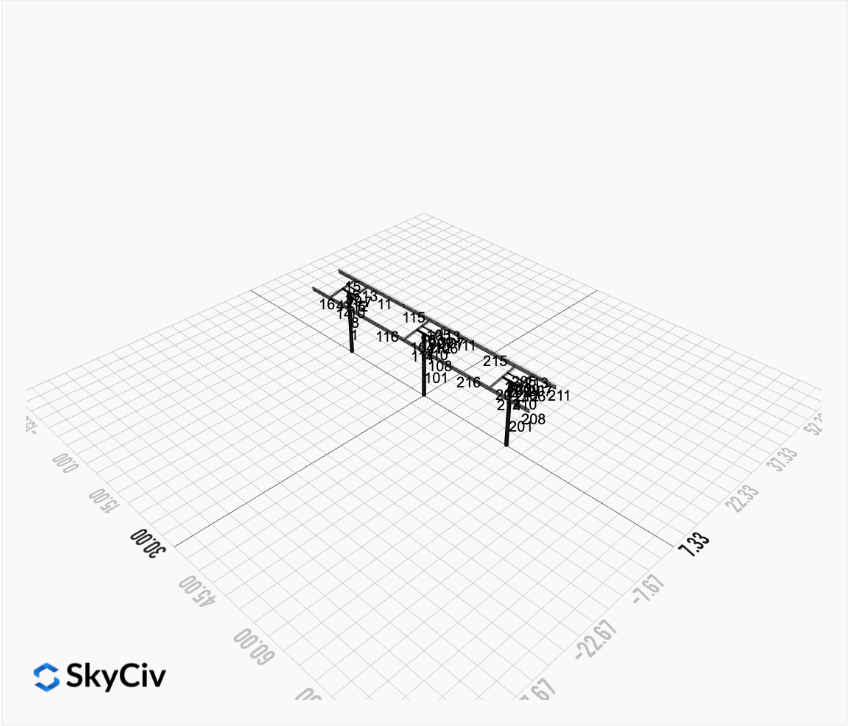
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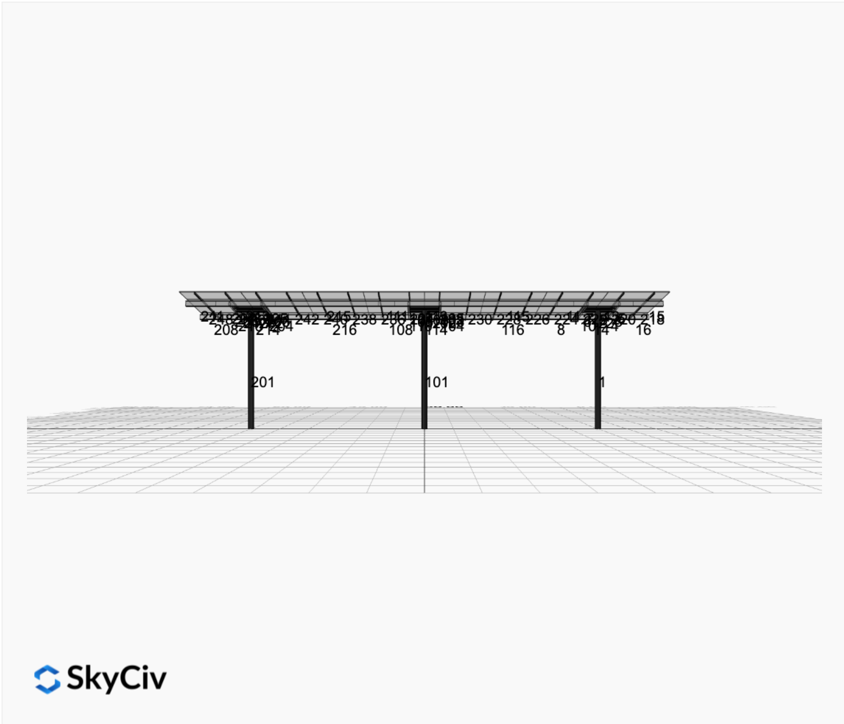
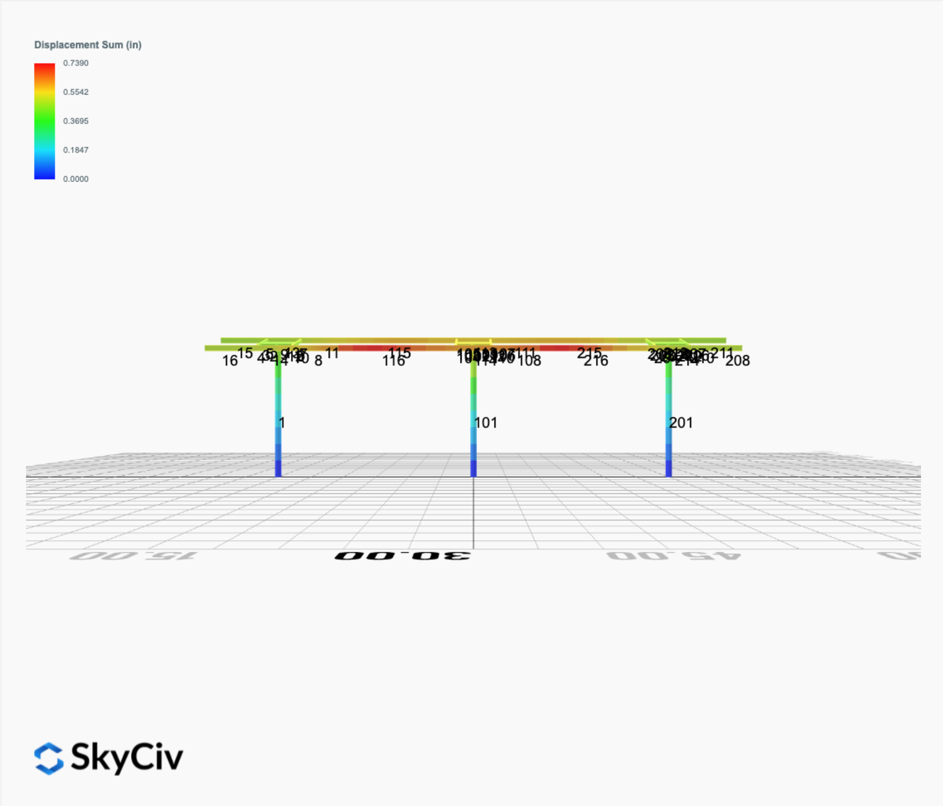
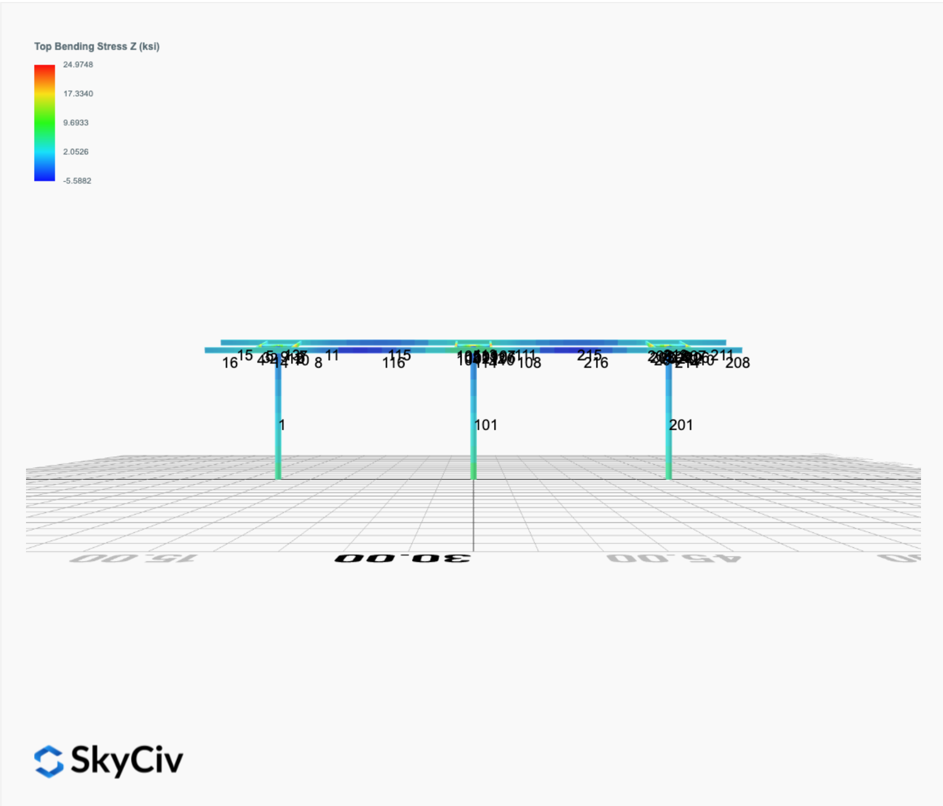
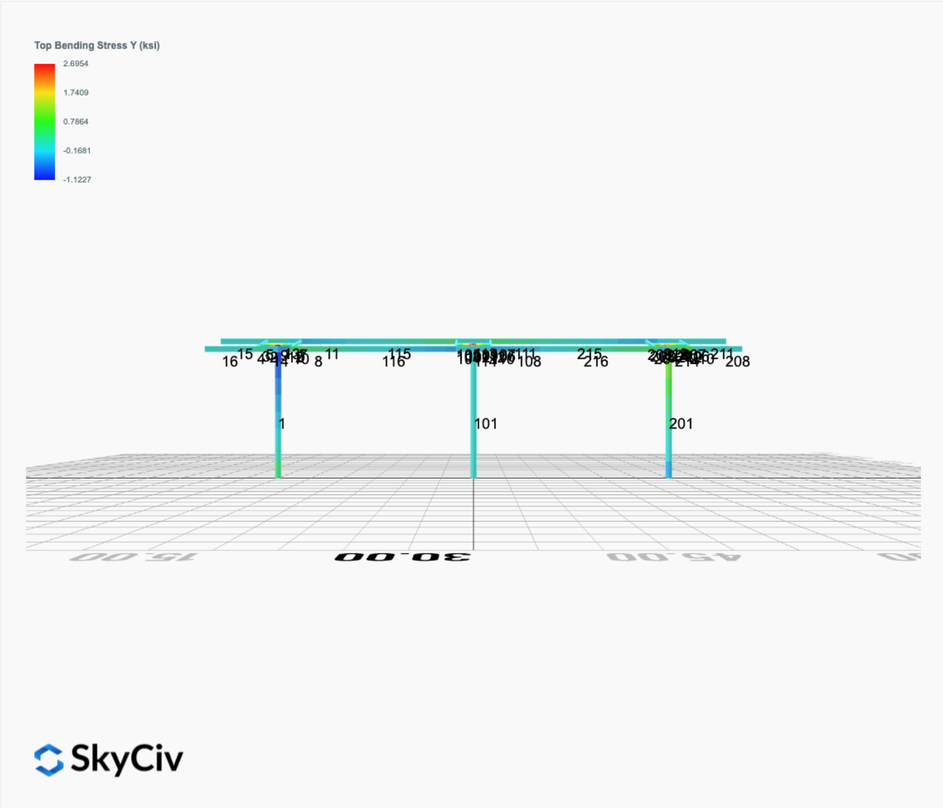
-22.67
SkyCiv


216
108
116
201
101
1

FEM Results (Envelope Worst Case for each member)






Top Bending Stress Y (ksi)
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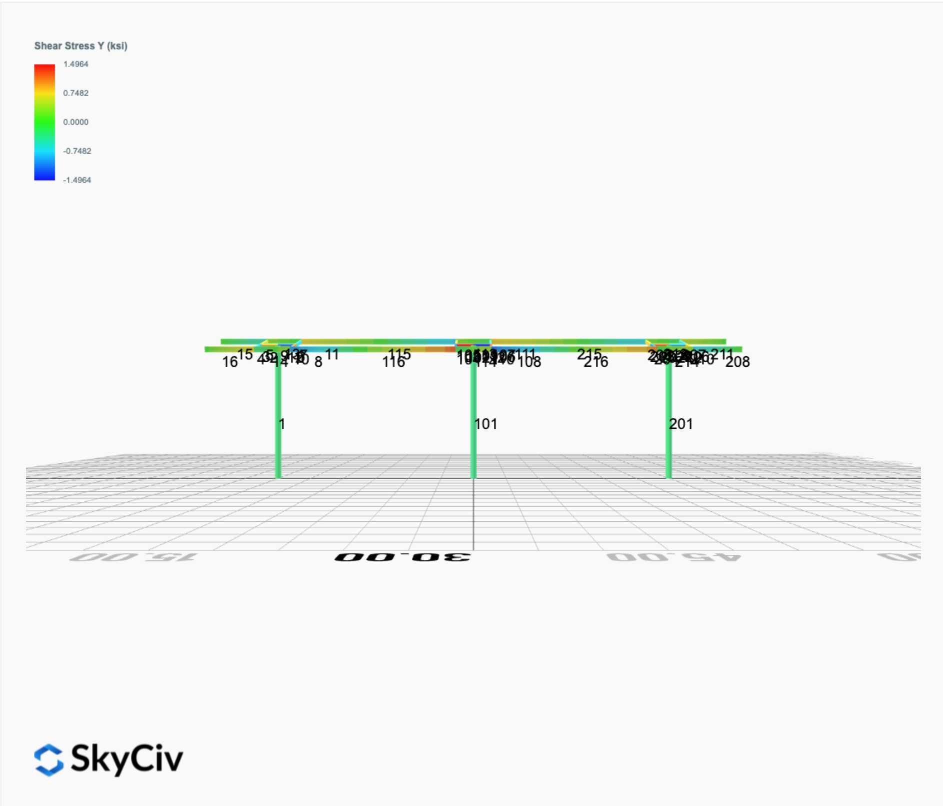
-1.1227
1
101
201
30.00
SkyCiv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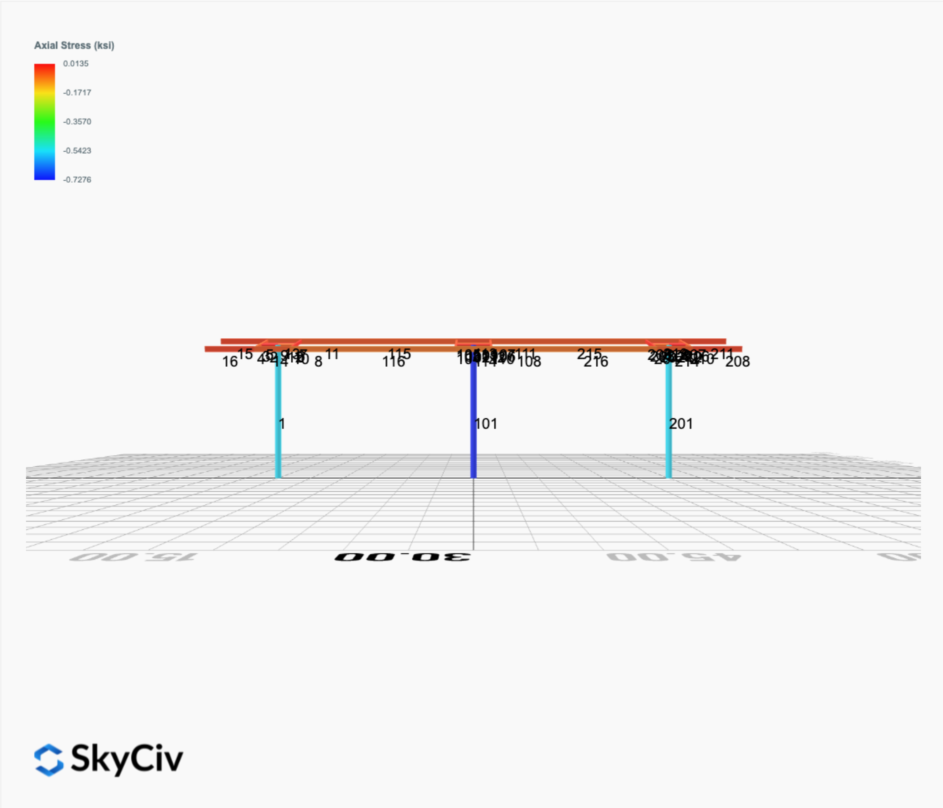
101
201
30.00
SkyCiv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15
8
11
115
116
108
215
216
208
1
101
201
SkyCiv

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	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 2. D + L	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 3. D + (S or Lr or R)	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 3. D + (S or Lr or R)	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 5b. D + 0.7E	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0071	2.2628	0.0502	0.2425	-0.0172	-0.0679
ULS: 8. 0.6D + 0.7E	0.0042	1.3577	0.0301	0.1455	-0.0103	-0.0408
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7243	6.2794	0.1872	0.9009	-0.1475	14.8736
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7243	6.2794	0.1872	0.9009	-0.1475	14.8736
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5092	-0.5479	-0.0394	-0.1859	0.0618	-5.2688
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4662	-0.1962	-0.0401	-0.1884	0.0763	-16.2531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5414	5.2752	0.1530	0.7363	-0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5414	5.2752	0.1530	0.7363	-0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3837	0.1548	-0.0170	-0.0788	0.0420	-3.9686
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3514	0.4185	-0.0175	-0.0807	0.0529	-12.2068
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5414	5.2752	0.1530	0.7363	-0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5414	5.2752	0.1530	0.7363	-0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3837	0.1548	-0.0170	-0.0788	0.0420	-3.9686
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3514	0.4185	-0.0175	-0.0807	0.0529	-12.2068
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7271	5.3743	0.1672	0.8039	-0.1406	14.9008
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7271	5.3743	0.1672	0.8039	-0.1406	14.9008
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5064	-1.4530	-0.0595	-0.2829	0.0686	-5.2416
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4634	-1.1013	-0.0602	-0.2854	0.0831	-16.2259

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	9.4103
Shear X	-1.2189
Shear Z	0.2897
Moment X	1.3956
Moment Y (Twist)	0.2393
Moment Z	27.5867

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	6.2794
Shear X	-0.7271
Shear Z	0.1872
Moment X	0.9009
Moment Y (Twist)	0.1475
Moment Z	16.2531

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	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 2. D + L	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 3. D + (S or Lr or R)	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 3. D + (S or Lr or R)	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 5b. D + 0.7E	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0141	2.7506	-0.0000	0.0000	0.0000	0.2239
ULS: 8. 0.6D + 0.7E	-0.0085	1.6504	-0.0000	0.0000	0.0000	0.1343
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8867	7.9613	0.0000	-0.0000	0.0000	18.0199
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8867	7.9613	0.0000	-0.0000	0.0000	18.0199
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6193	-0.9161	0.0000	-0.0000	0.0000	-6.2233
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4931	-0.4154	-0.0000	0.0000	0.0000	-18.4070
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6686	6.6586	0.0000	-0.0000	0.0000	13.5709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6686	6.6586	0.0000	-0.0000	0.0000	13.5709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4610	0.0006	0.0000	-0.0000	0.0000	-4.6115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3663	0.3761	-0.0000	0.0000	0.0000	-13.7493
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6686	6.6586	0.0000	-0.0000	0.0000	13.5709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6686	6.6586	0.0000	-0.0000	0.0000	13.5709
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4610	0.0006	0.0000	-0.0000	0.0000	-4.6115
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3663	0.3761	-0.0000	0.0000	0.0000	-13.7493
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8811	6.8610	0.0000	-0.0000	0.0000	17.9303
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8811	6.8610	0.0000	-0.0000	0.0000	17.9303
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6250	-2.0163	0.0000	-0.0000	0.0000	-6.3128
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4987	-1.5157	-0.0000	0.0000	0.0000	-18.4966

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	11.9840
Shear X	-1.4688
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	31.4438

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	7.9613
Shear X	-0.8867
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	18.4966

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	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 2. D + L	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 3. D + (S or Lr or R)	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 3. D + (S or Lr or R)	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 5b. D + 0.7E	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0071	2.2628	-0.0502	-0.2425	0.0172	-0.0679
ULS: 8. 0.6D + 0.7E	0.0042	1.3577	-0.0301	-0.1455	0.0103	-0.0408
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7243	6.2794	-0.1872	-0.9009	0.1475	14.8737
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7243	6.2794	-0.1872	-0.9009	0.1475	14.8737
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5092	-0.5479	0.0394	0.1859	-0.0618	-5.2688
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4662	-0.1962	0.0401	0.1884	-0.0763	-16.2531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5414	5.2752	-0.1530	-0.7363	0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5414	5.2752	-0.1530	-0.7363	0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3837	0.1548	0.0170	0.0788	-0.0420	-3.9686
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3514	0.4185	0.0175	0.0807	-0.0529	-12.2068

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	-0.5414	5.2752	-0.1530	-0.7363	0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5414	5.2752	-0.1530	-0.7363	0.1149	11.1383
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3837	0.1548	0.0170	0.0788	-0.0420	-3.9686
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3514	0.4185	0.0175	0.0807	-0.0529	-12.2068
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7271	5.3743	-0.1672	-0.8039	0.1406	14.9008
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7271	5.3743	-0.1672	-0.8039	0.1406	14.9008
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5064	-1.4530	0.0595	0.2829	-0.0686	-5.2416
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4634	-1.1013	0.0602	0.2854	-0.0831	-16.2259

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	9.4103
Shear X	-1.2189
Shear Z	-0.2897
Moment X	-1.3957
Moment Y (Twist)	0.2393
Moment Z	27.5869

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	6.2794
Shear X	-0.7271
Shear Z	-0.1872
Moment X	-0.9009
Moment Y (Twist)	0.1475
Moment Z	16.2531

Project Details

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

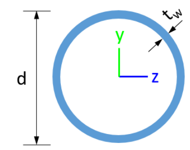
User Name: sales@mtsolar.us
Project Name: Tavoli Apartments-cuV1
Unit System: imperial

Design Input Information

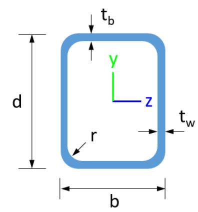
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

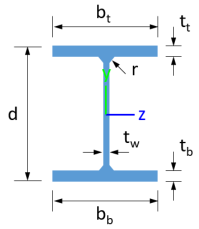
Section Dimensions



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	32.14	32.14	15.31	-	300	200	1
2	5	1.30	1.30	2.00	-	300	200	1
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.29,1.16,1.18,1.18,1.29,1.16,1.17,1.17,1.17,1.17	300	200	1
4	16	2.44	2.44	3.75	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.68,1.68,1.62,1.69,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.68	300	200	1
5	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.86,1.65,1.67,1.67,1.86,1.65,1.67,1.67,1.66,1.66	300	200	1
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.35,1.18,1.18,1.18,1.35,1.18,1.18,1.18,1.18,1.18	300	200	1
7	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.98,1.66,1.67,1.67,1.98,1.66,1.67,1.67,1.66,1.66	300	200	1
8	19	1.33	1.33	2.05	1.67,1.67,1.67,1.67,1.67,1.67,1.62,1.62,1.72,2.12,1.62,1.62,1.72,2.12,1.63,1.63,1.76,2.03,1.63,1.63,1.76,2.03,1.62,1.62,1.71,2.22	300	200	1
9	2	2.60	2.60	4.00	-	300	200	1
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.66,1.68	300	200	1
11	19	1.33	1.33	2.05	1.72,1.72,1.72,1.72,1.72,1.72,1.80,1.80,1.66,1.99,1.80,1.80,1.66,1.99,1.78,1.78,2.24,2.08,1.78,1.78,2.24,2.08,1.80,1.80,1.67,1.97	300	200	1
12	5	4.20	4.20	2.00	-	300	200	1
13	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.06,1.11,1.09,1.09,1.06,1.11,1.09,1.09,2.27,1.18,1.09,1.09,2.27,1.18,1.09,1.09,1.06,1.11	300	200	1
14	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.08,1.10,1.40,1.08,1.08,1.10,1.40,1.08,1.08,1.12,1.16,1.08,1.08,1.12,1.16,1.07,1.07,1.10,1.56	300	200	1
15	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33	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	32.14	32.14	15.31	-	300	200	1
102	5	1.30	1.30	2.00	-	300	200	1
103	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.31,1.17,1.18,1.18,1.31,1.17,1.18,1.18,1.17,1.18	300	200	1
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.68	300	200	1
105	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.89,1.66,1.67,1.67,1.89,1.66,1.67,1.67,1.66,1.66	300	200	1
106	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.31,1.17,1.18,1.18,1.31,1.17,1.18,1.18,1.17,1.18	300	200	1
107	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.89,1.66,1.67,1.67,1.89,1.66,1.67,1.67,1.66,1.66	30	20	1

						0	0	
108	19	1.33	1.33	2.05	2.23,2.23,2.23,2.23,2.23,2.23,2.18,2.18,2.26,1.62,2.18,2.18,2.26,1.62,2.21,2.21,2.28,2.11,2.21,2.21,2.28,2.11,2.17,2.17,2.25,1.42	300	200	1
109	2	2.60	2.60	4.00	-	300	200	1
110	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.66,1.68	300	200	1
111	19	1.33	1.33	2.05	2.25,2.25,2.25,2.25,2.25,2.25,2.33,2.33,2.24,2.11,2.33,2.33,2.24,2.11,2.31,2.31,1.84,1.99,2.31,2.31,1.84,1.99,2.34,2.34,2.24,2.11	300	200	1
112	5	1.30	1.30	2.00	-	300	200	1
113	19	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.03,1.02,1.04,1.03,1.03,1.02,1.04,1.03,1.03,2.08,1.09,1.03,1.03,2.08,1.09,1.03,1.03,1.02,1.03	300	200	1
114	19	4.88	4.00	7.50	1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.02,1.03,1.40,1.02,1.02,1.03,1.40,1.02,1.02,1.03,1.08,1.02,1.02,1.03,1.08,1.02,1.02,1.03,2.25	300	200	1
115	19	8.42	8.42	12.95	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.19,1.14,1.17,1.17,1.19,1.14,1.17,1.17,1.13,1.13,1.17,1.17,1.13,1.13,1.17,1.17,1.19,1.15	300	200	1
116	19	8.42	8.42	12.95	1.19,1.19,1.19,1.19,1.19,1.19,1.20,1.20,1.18,1.11,1.20,1.20,1.18,1.11,1.20,1.20,1.18,1.14,1.20,1.20,1.18,1.14,1.20,1.20,1.18,1.11	300	200	1
201	9	32.14	32.14	15.31	-	300	200	1
202	5	4.20	4.20	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.35,1.18,1.18,1.18,1.35,1.18,1.18,1.18,1.18,1.18	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.63,1.68,1.67,1.67,1.66,1.68	300	200	1
205	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.98,1.66,1.67,1.67,1.98,1.66,1.67,1.67,1.66,1.66	300	200	1
206	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.16,1.17,1.18,1.18,1.29,1.16,1.18,1.18,1.29,1.16,1.17,1.17,1.17,1.17	300	200	1
207	16	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.65,1.66,1.67,1.67,1.65,1.66,1.67,1.67,1.86,1.65,1.67,1.67,1.86,1.65,1.67,1.67,1.66,1.66	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	1.69,1.69,1.69,1.69,1.69,1.69,1.67,1.67,1.66,1.68,1.67,1.67,1.66,1.68,1.68,1.68,1.62,1.69,1.68,1.68,1.62,1.69,1.67,1.67,1.66,1.68	300	200	1
211	19	7.88	7.88	3.75	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.32,2.33,2.33,2.33,2.33,2.33	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.06,1.11,1.09,1.09,1.06,1.11,1.09,1.09,2.27,1.18,1.09,1.09,2.27,1.18,1.09,1.09,1.06,1.11	300	200	1
214	19	4.88	4.00	7.50	1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.08,1.10,1.40,1.08,1.08,1.10,1.40,1.08,1.08,1.12,1.16,1.08,1.08,1.12,1.16,1.07,1.07,1.10,1.56	300	200	1
215	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.12,1.12,1.12,1.13,1.12,1.12,1.12,1.13,1.12,1.12,1.19,1.13,1.12,1.12,1.19,1.13,1.12,1.12,1.12,1.12	300	200	1
216	19	8.42	8.42	12.95	1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.13,1.15,1.13,1.13,1.13,1.15,1.13,1.13,1.12,1.13,1.13,1.13,1.12,1.13,1.13,1.13,1.13,1.17	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	110.06	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	182.14	21.95	21.95	59.50	59.50
13	133.20	85.85	24.21	6.12	40.24	43.62
14	133.20	85.85	24.62	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	110.06	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	23.44	6.12	40.24	43.62
114	133.20	85.85	23.42	6.12	40.24	43.62
115	133.20	46.28	12.20	6.12	40.24	43.62
116	133.20	46.28	11.99	6.12	40.24	43.62
201	377.97	110.06	83.29	83.29	113.39	113.39
202	198.33	182.14	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	24.21	6.12	40.24	43.62
214	133.20	85.85	24.62	6.12	40.24	43.62
215	133.20	46.28	12.16	6.12	40.24	43.62
216	133.20	46.28	12.18	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	0.085	0.331	0.036	0.011	0.003	0.363	#13	0.657	Not Required	Pass
2	0.001	0.288	0.048	0.065	0.009	0.336	#13	0.035	Not Required	Pass
3	0.001	0.531	0.007	0.052	0.002	0.536	#13	0.045	Not Required	Pass
4	0.001	0.383	0.024	0.039	0.006	0.408	#13	0.080	Not Required	Pass
5	0.001	0.329	0.013	0.053	0.004	0.334	#13	0.074	Not Required	Pass
6	0.001	0.683	0.016	0.070	0.004	0.699	#13	0.045	Not Required	Pass
7	0.001	0.423	0.029	0.068	0.007	0.431	#13	0.074	Not Required	Pass
8	0.002	0.070	0.045	0.034	0.002	0.092	#13	0.095	Not Required	Pass
9	0.002	0.082	0.021	0.003	0.001	0.103	#13	0.204	Not Required	Pass
10	0.001	0.494	0.035	0.050	0.008	0.508	#13	0.080	Not Required	Pass
11	0.001	0.087	0.044	0.049	0.002	0.106	#13	0.095	Not Required	Pass
12	0.001	0.426	0.058	0.085	0.012	0.485	#13	0.171	Not Required	Pass
13	0.001	0.196	0.072	0.061	0.002	0.213	#13	0.286	Not Required	Pass
14	0.002	0.142	0.073	0.043	0.002	0.159	#13	0.190	Not Required	Pass
15	0.000	0.059	0.012	0.026	0.001	0.070	#13	Not Required	Not Required	Pass
16	0.000	0.043	0.012	0.019	0.001	0.054	#13	Not Required	Not Required	Pass
101	0.109	0.378	0.000	0.013	0.000	0.421	#13	0.657	Not Required	Pass
102	0.001	0.465	0.068	0.096	0.012	0.534	#13	0.053	Not Required	Pass
103	0.001	0.769	0.009	0.078	0.002	0.773	#13	0.045	Not Required	Pass
104	0.001	0.583	0.032	0.059	0.006	0.602	#13	0.080	Not Required	Pass
105	0.001	0.477	0.033	0.077	0.009	0.487	#13	0.074	Not Required	Pass
106	0.001	0.769	0.009	0.078	0.002	0.773	#13	0.045	Not Required	Pass
107	0.001	0.477	0.033	0.077	0.009	0.487	#13	0.074	Not Required	Pass
108	0.002	0.072	0.046	0.042	0.002	0.118	#13	0.095	Not Required	Pass
109	0.004	0.077	0.015	0.001	0.000	0.094	#13	0.204	Not Required	Pass
110	0.001	0.583	0.032	0.059	0.006	0.602	#13	0.080	Not Required	Pass
111	0.001	0.067	0.045	0.054	0.002	0.113	#13	0.095	Not Required	Pass
112	0.001	0.465	0.068	0.096	0.012	0.534	#13	0.053	Not Required	Pass
113	0.001	0.290	0.073	0.066	0.002	0.347	#13	0.286	Not Required	Pass
114	0.004	0.261	0.074	0.050	0.002	0.314	#13	0.286	Not Required	Pass
115	0.002	0.498	0.045	0.054	0.002	0.530	#13	0.601	Not Required	Pass
116	0.004	0.353	0.046	0.042	0.002	0.377	#13	0.601	Not Required	Pass
201	0.085	0.331	0.036	0.011	0.003	0.363	#13	0.657	Not Required	Pass
202	0.001	0.426	0.058	0.085	0.012	0.485	#13	0.171	Not Required	Pass
203	0.001	0.683	0.016	0.070	0.004	0.699	#13	0.045	Not Required	Pass
204	0.001	0.494	0.035	0.050	0.008	0.508	#13	0.080	Not Required	Pass
205	0.001	0.423	0.029	0.068	0.007	0.431	#13	0.074	Not Required	Pass
206	0.001	0.531	0.007	0.052	0.002	0.536	#13	0.045	Not Required	Pass
207	0.001	0.329	0.013	0.053	0.004	0.334	#13	0.074	Not Required	Pass
208	0.000	0.043	0.012	0.019	0.001	0.054	#13	Not Required	Not Required	Pass
209	0.002	0.082	0.021	0.003	0.001	0.103	#13	0.204	Not Required	Pass
210	0.001	0.383	0.024	0.039	0.006	0.408	#13	0.120	Not Required	Pass
211	0.000	0.059	0.012	0.026	0.001	0.070	#13	Not Required	Not Required	Pass
212	0.001	0.288	0.048	0.065	0.009	0.336	#13	0.035	Not Required	Pass
213	0.001	0.196	0.072	0.061	0.002	0.213	#13	0.190	Not Required	Pass
214	0.002	0.142	0.073	0.043	0.002	0.159	#13	0.286	Not Required	Pass
215	0.002	0.520	0.044	0.049	0.002	0.547	#13	0.601	Not Required	Pass
216	0.004	0.373	0.045	0.034	0.002	0.401	#13	0.601	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/comression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P,M_z,M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS

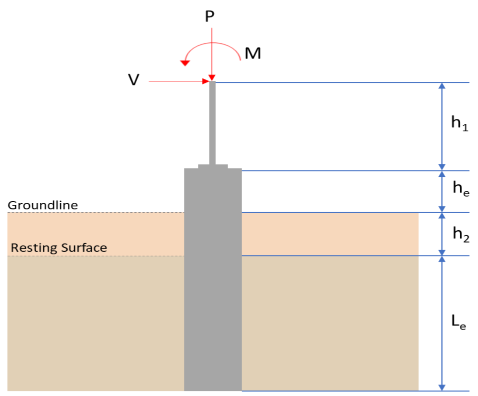
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 5.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.279	9.410
V_x (kip)	-0.727	-1.219
V_z (kip)	0.187	0.290
M_x (kipft)	0.901	1.396
M_z (kipft)	16.253	27.587

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.727\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.11576\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.253\ \text{kipft}) + ((-0.727\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 2.5881\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.5252\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.187\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.029777\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.901\ \text{kipft}) + ((0.187\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.14347\ \text{kipft/ft}$$

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:
$L_{e,z} = 2.5187\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.5252\ \text{ft}), (2.5187\ \text{ft})]$$

$$L_{e,req} = 5.525\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 5.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.525\ \text{ft})}{(5.75\ \text{ft})}$$

$$Ratio = 0.96087$$

Status: **PASS**
Ratio: **0.960**

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

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

$$q = \frac{(6.279\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.39244\ \text{kip/ft}^2$$

$q = 0.39244\ \text{kip/ft}^2$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39244\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.19622$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.11576\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.5881\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5881\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.11576\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (2.5881\ \text{kipft/ft})) + (4 \times (-0.11576\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9035\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.5881\ \text{kipft/ft})) + (3 \times (-0.11576\ \text{kip/ft}) \times (5.75\ \text{ft}))]^2}{(5.75\ \text{ft})^2 \times [(3 \times (2.5881\ \text{kipft/ft})) + (2 \times (-0.11576\ \text{kip/ft}) \times (5.75\ \text{ft}))]}$$

$$p = 0.24617\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.5881\ \text{kipft/ft})) + ((-0.11576\ \text{kip/ft}) \times (5.75\ \text{ft}))]}{(5.75\ \text{ft})^2}$$

$$s = 0.81854\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(3.9035\ \text{ft})}{2}$$

$$p_a = 0.29276\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.24617\ \text{kip/ft}^2)}{(0.29276\ \text{kip/ft}^2)}$$

$$Ratio = 0.84088$$

Status: **PASS**
Ratio: **0.840**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \text{psf/ft}) \times (5.75\ \text{ft})$$

$$p_s = 0.8625\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.81854\ \text{kip/ft}^2\right)}{\left(0.8625\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.94903$$

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

Considering z-direction:

$H_o = 0.029777\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.14347\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14347\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (0.029777\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (0.14347\ \text{kipft/ft})) + (4 \times (0.029777\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 4.0456\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.14347\ \text{kipft/ft})) + (3 \times (0.029777\ \text{kip/ft}) \times (5.75\ \text{ft}))]^{\ 2}}{(5.75\ \text{ft})^2 \times [(3 \times (0.14347\ \text{kipft/ft})) + (2 \times (0.029777\ \text{kip/ft}) \times (5.75\ \text{ft}))]}$$

$$p = 0.034715\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.14347\ \text{kipft/ft})) + ((0.029777\ \text{kip/ft}) \times (5.75\ \text{ft}))]}{(5.75\ \text{ft})^2}$$

$$s = 0.083144\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.0456\ \text{ft})}{2}$$

$$p_a = 0.30342\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.034715\ \text{kip/ft}^2\right)}{\left(0.30342\ \text{kip/ft}^2\right)}$$

$$Ratio = 0.11441$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \text{psf/ft}) \times (5.75\ \text{ft})$$

$$p_s = 0.8625\ \text{kip/ft}^2$$

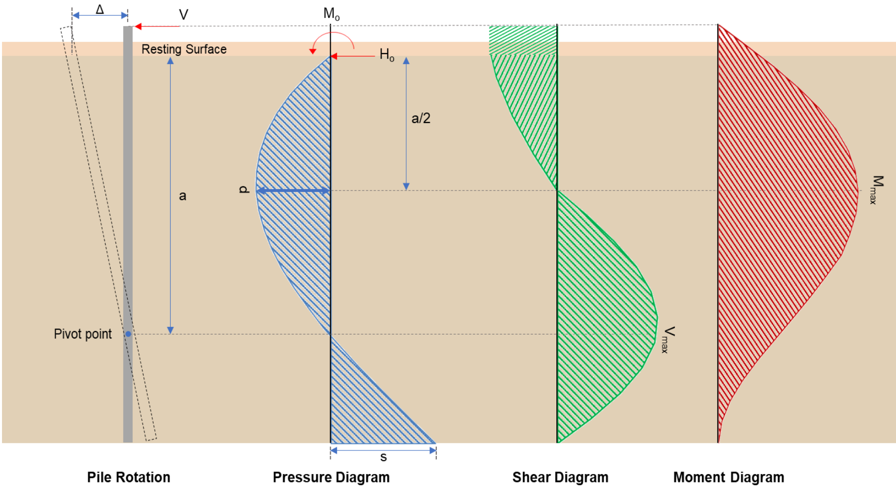
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.083144\ \text{kip/ft}^2)}{(0.8625\ \text{kip/ft}^2)}$$

$$Ratio = 0.0964$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.219\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.19411\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.587\ \text{kipft}) + ((-1.219\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.3928\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3928\ \text{kipft/ft})}{(-0.19411\ \text{kip/ft})}$$

$$E = 22.631\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3928\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.19411\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (4.3928\ \text{kipft/ft})) + (4 \times (-0.19411\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9027\ \text{ft}$$

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

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

$$V_{max} = ((-0.19411\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.631\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9027\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (22.631\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9027\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 5.9278\ \text{kip}$$

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

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

$$M_{max} = ((-0.19411\text{ kip/ft}) \times (48\text{ in}) \times (5.75\text{ ft})) \times \left[\left(\frac{(22.631\text{ ft})}{(5.75\text{ ft})} + \frac{(3.9027\text{ ft})}{2 \times (5.75\text{ ft})}\right) - \left[\left(\frac{4 \times (22.631\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(3.9027\text{ ft})}{2 \times (5.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (22.631\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(3.9027\text{ ft})}{2 \times (5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 16.633\text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.29\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = 0.046178\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.396\text{ kipft}) + ((0.29\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 0.22229\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.22229\text{ kipft/ft})}{(0.046178\text{ kip/ft})}$$

$$E = 4.8138\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.22229\text{ kipft/ft}) \times (5.75\text{ ft})) + \left(3 \times (0.046178\text{ kip/ft}) \times (5.75\text{ ft})^2\right)}{(6 \times (0.22229\text{ kipft/ft})) + (4 \times (0.046178\text{ kip/ft}) \times (5.75\text{ ft}))}$$

$$a = 4.0458\text{ ft}$$

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

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

$$V_{max} = ((0.046178\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.8138\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(4.0458\text{ ft})}{(5.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.8138\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(4.0458\text{ ft})}{(5.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.39585\text{ kip}$$

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

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

$$M_{max} = ((0.046178\text{ kip/ft}) \times (48\text{ in}) \times (5.75\text{ ft})) \times \left[\left(\frac{(4.8138\text{ ft})}{(5.75\text{ ft})} + \frac{(4.0458\text{ ft})}{2 \times (5.75\text{ ft})}\right) - \left[\left(\frac{4 \times (4.8138\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(4.0458\text{ ft})}{2 \times (5.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.8138\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(4.0458\text{ ft})}{2 \times (5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 1.0426 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.41 \text{ kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2))\right]$$

$$A_{st,required} = -84.283 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.283 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: **#3(0.375 in) - 10 in**

22.4.2.2	**Axial Compression Strength (ACI 318-19, LRFD)** ϕP_N - Allowable axial compressive strength $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\text{ ksi}) \times \left[\left(2304\text{ in}^2\right) - \left(4.2951\text{ in}^2\right)\right]\right) + \left((60\text{ ksi}) \times \left(4.2951\text{ in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\text{ kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(9.41\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0035175$$	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 $f'_{ck} = 2.5\text{ ksi} \to 2500\text{ psi}$, $V_{c,max}$ - 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 $f'_{ck} = 2.5\text{ ksi} \to 2500\text{ psi}$, $P = 9.41\text{ kip} \to 9410\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(9410\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 119.74\text{ kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \to 2500\text{ psi}$, $V_{c,b}$ - Shear strength of concrete (b) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right]\, b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (119.74\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 119.74\text{ kip}$$	

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

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

$$\phi V_n = (0.65) \times ((119.74\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 110.91\ \text{kip}$$

Considering x-direction:

$V_{max} = 5.9278\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(5.9278\ \text{kip})}{(110.91\ \text{kip})}$$

$$Ratio = 0.053446$$

Status: **PASS**
Ratio: **0.050**

Considering z-direction:

$V_{max} = 0.39585\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39585\ \text{kip})}{(110.91\ \text{kip})}$$

$$Ratio = 0.003569$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f_{ck}'\, S_m$$ $$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times \left(18432 \text{ in}^3\right)$$ $$\phi M_{n,2} = 2121.6 \text{ kipft}$$ Therefore, ϕM_n - Allowable flexural strength, $$\phi M_n = MIN[\phi M_{n,1}, \phi M_{n,2}]$$ $$\phi M_n = MIN[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$$ $$\phi M_n = 249.6 \text{ kipft}$$ **Considering x-direction:** $M_{max} = 16.633 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(16.633 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.066641$$	Status: **PASS** Ratio: **0.070**
	Considering z-direction: $M_{max} = 1.0426 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(1.0426 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0041772$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

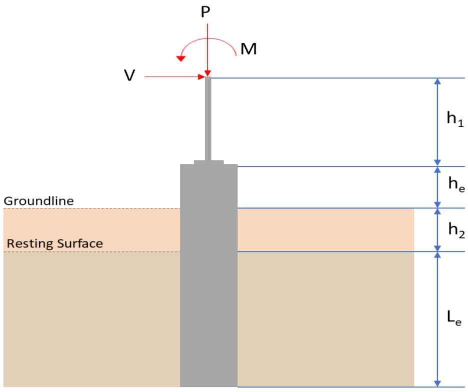
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 5.75$ 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)	6.279	9.410
V_x (kip)	-0.727	-1.219
V_z (kip)	-0.187	-0.290
M_x (kipft)	-0.901	-1.396
M_z (kipft)	16.253	27.587

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,

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

$$H_o = \frac{(-0.727 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11576 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.253 \text{ kipft}) + ((-0.727 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.5881 \text{ kipft/ft}$$

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:
$L_{e,x} = 5.5252 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.187 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.029777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.901 \text{ kipft}) + ((-0.187 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14347 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,z} = 1.993 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.5252 \text{ ft}), (1.993 \text{ ft})]$$

$$L_{e,req} = 5.525 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.525 \text{ ft})}{(5.75 \text{ ft})}$$

$$Ratio = 0.96087$$

Status: **PASS**
Ratio: **0.960**

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

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

$$q = \frac{(6.279 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39244 \text{ kip/ft}^2$$

$q = 0.39244\ \mathrm{kip/ft^2}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.39244\ \mathrm{kip/ft^2}\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.19622$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.11576\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.5881\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.5881\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (-0.11576\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (2.5881\ \mathrm{kipft/ft})) + (4 \times (-0.11576\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 3.9035\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.5881\ \mathrm{kipft/ft})) + (3 \times (-0.11576\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]^2}{(5.75\ \mathrm{ft})^2 \times [(3 \times (2.5881\ \mathrm{kipft/ft})) + (2 \times (-0.11576\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}$$

$$p = 0.24617\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.5881\ \mathrm{kipft/ft})) + ((-0.11576\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}{(5.75\ \mathrm{ft})^2}$$

$$s = 0.81854\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(3.9035\ \mathrm{ft})}{2}$$

$$p_a = 0.29276\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.24617\ \mathrm{kip/ft^2}\right)}{\left(0.29276\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.84088$$

Status: **PASS**
Ratio: **0.840**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 0.8625\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.81854\ \mathrm{kip/ft^2}\right)}{\left(0.8625\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.94903$$

Status: PASS
Ratio: 0.950

Considering z-direction:

$H_o = -0.029777\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.14347\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14347\ \mathrm{kipft/ft}) \times (5.75\ \mathrm{ft})) + \left(3 \times (-0.029777\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft})^2\right)}{(6 \times (0.14347\ \mathrm{kipft/ft})) + (4 \times (-0.029777\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))}$$

$$a = 4.0456\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.14347\ \mathrm{kipft/ft})) + (3 \times (-0.029777\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]^2}{(5.75\ \mathrm{ft})^2 \times [(3 \times (0.14347\ \mathrm{kipft/ft})) + (2 \times (-0.029777\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}$$

$$p = 0.00093539\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.14347\ \mathrm{kipft/ft})) + ((-0.029777\ \mathrm{kip/ft}) \times (5.75\ \mathrm{ft}))]}{(5.75\ \mathrm{ft})^2}$$

$$s = 0.021001\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.0456\ \mathrm{ft})}{2}$$

$$p_a = 0.30342\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.00093539\ \mathrm{kip/ft^2}\right)}{\left(0.30342\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.0030828$$

Status: PASS
Ratio: 0.000

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (5.75\ \mathrm{ft})$$

$$p_s = 0.8625\ \mathrm{kip/ft^2}$$

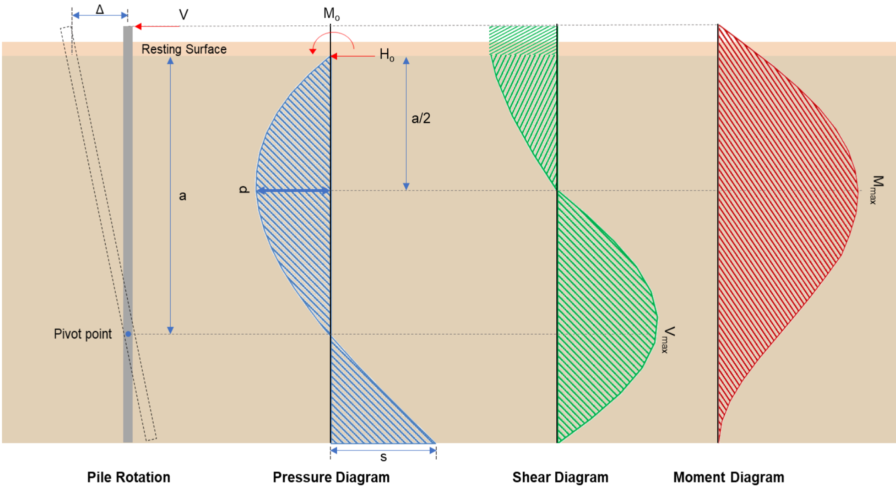
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.021001\ \text{kip/ft}^2)}{(0.8625\ \text{kip/ft}^2)}$$

$$Ratio = 0.024349$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.219\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.19411\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.587\ \text{kipft}) + ((-1.219\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.3928\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.3928\ \text{kipft/ft})}{(-0.19411\ \text{kip/ft})}$$

$$E = 22.631\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.3928\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.19411\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (4.3928\ \text{kipft/ft})) + (4 \times (-0.19411\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9027\ \text{ft}$$

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

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

$$V_{max} = ((-0.19411\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.631\ \text{ft})}{(5.75\ \text{ft})} + 3\right) \times \left(\frac{(3.9027\ \text{ft})}{(5.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (22.631\ \text{ft})}{(5.75\ \text{ft})} + 2\right) \times \left(\frac{(3.9027\ \text{ft})}{(5.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 5.9278\ \text{kip}$$

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

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

$$M_{max} = ((-0.19411 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(22.631 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9027 \text{ ft})}{2 \times (5.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (22.631 \text{ ft})}{(5.75 \text{ ft})} + 3\right) \times \left(\frac{(3.9027 \text{ ft})}{2 \times (5.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (22.631 \text{ ft})}{(5.75 \text{ ft})} + 2\right) \times \left(\frac{(3.9027 \text{ ft})}{2 \times (5.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 16.633 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.29 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.396 \text{ kipft}) + ((-0.29 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.22229 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.22229 \text{ kipft/ft})}{(-0.046178 \text{ kip/ft})}$$

$$E = 4.8138 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.22229 \text{ kipft/ft}) \times (5.75 \text{ ft})) + \left(3 \times (-0.046178 \text{ kip/ft}) \times (5.75 \text{ ft})^2\right)}{(6 \times (0.22229 \text{ kipft/ft})) + (4 \times (-0.046178 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0458 \text{ ft}$$

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

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

$$V_{max} = ((-0.046178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.8138 \text{ ft})}{(5.75 \text{ ft})} + 3\right) \times \left(\frac{(4.0458 \text{ ft})}{(5.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (4.8138 \text{ ft})}{(5.75 \text{ ft})} + 2\right) \times \left(\frac{(4.0458 \text{ ft})}{(5.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 0.39585 \text{ kip}$$

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

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

$$M_{max} = ((-0.046178 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.8138 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.0458 \text{ ft})}{2 \times (5.75 \text{ ft})}\right) - \left[\left(\frac{4 \times (4.8138 \text{ ft})}{(5.75 \text{ ft})} + 3\right) \times \left(\frac{(4.0458 \text{ ft})}{2 \times (5.75 \text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (4.8138 \text{ ft})}{(5.75 \text{ ft})} + 2\right) \times \left(\frac{(4.0458 \text{ ft})}{2 \times (5.75 \text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 1.0426 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(9.41 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.283 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.283 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(9.41\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0035175$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 9.41\ \text{kip} \rightarrow 9410\ \text{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(9410\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.74\ \text{kip}$$

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

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.74\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.74\ \text{kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.74 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.91 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.9278 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(5.9278 \text{ kip})}{(110.91 \text{ kip})}$$

$$Ratio = 0.053446$$

Status: **PASS**
Ratio: **0.050**

Considering z-direction:

$V_{max} = 0.39585 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39585 \text{ kip})}{(110.91 \text{ kip})}$$

$$Ratio = 0.003569$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

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

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

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

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

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

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

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

Therefore,
ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 16.633 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(16.633 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.066641$$

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

Considering z-direction:

$M_{max} = 1.0426 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(1.0426 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0041772$$

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

REFERENCES	CALCULATIONS	RESULTS

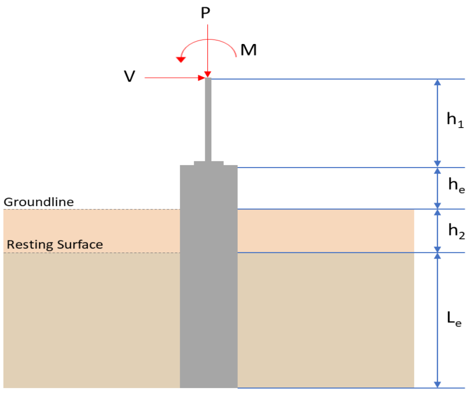
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48$ in - Pile width
$D = 48$ in - Pile depth
$L = 6$ 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)	7.961	11.984
V_x (kip)	-0.887	-1.469
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	18.497	31.444

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,

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

$$H_o = \frac{(-0.887 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14124 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.497\ \text{kipft}) + ((-0.887\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 2.9454\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.7201\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.7201\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 5.72\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.72\ \text{ft})}{(6\ \text{ft})}$$

$$Ratio = 0.95333$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.961\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.49756\ \text{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.49756\ \text{kip/ft}^2\right)}{(2000\ \text{psf})}$$

$$Ratio = 0.24878$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.5$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.14124\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 2.9454\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9454\ \mathrm{kipft/ft}) \times (6\ \mathrm{ft})) + \left(3 \times (-0.14124\ \mathrm{kip/ft}) \times (6\ \mathrm{ft})^2\right)}{(6 \times (2.9454\ \mathrm{kipft/ft})) + (4 \times (-0.14124\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))}$$

$$a = 4.0805\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9454\ \mathrm{kipft/ft})) + (3 \times (-0.14124\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]^2}{(6\ \mathrm{ft})^2 \times [(3 \times (2.9454\ \mathrm{kipft/ft})) + (2 \times (-0.14124\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}$$

$$p = 0.24903\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (2.9454\ \mathrm{kipft/ft})) + ((-0.14124\ \mathrm{kip/ft}) \times (6\ \mathrm{ft}))]}{(6\ \mathrm{ft})^2}$$

$$s = 0.84055\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.0805\ \mathrm{ft})}{2}$$

$$p_a = 0.30604\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.24903\ \mathrm{kip/ft^2})}{(0.30604\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.81373$$

Status: **PASS**
Ratio: **0.810**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6\ \mathrm{ft})$$

$$p_s = 0.9\ \mathrm{kip/ft^2}$$

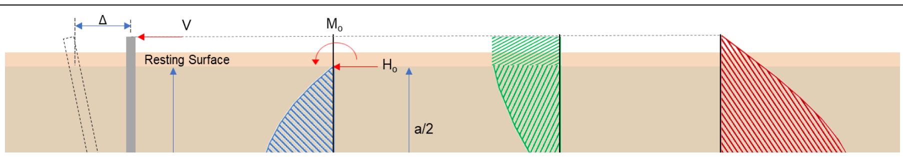
$Ratio$ - Lateral soil capacity

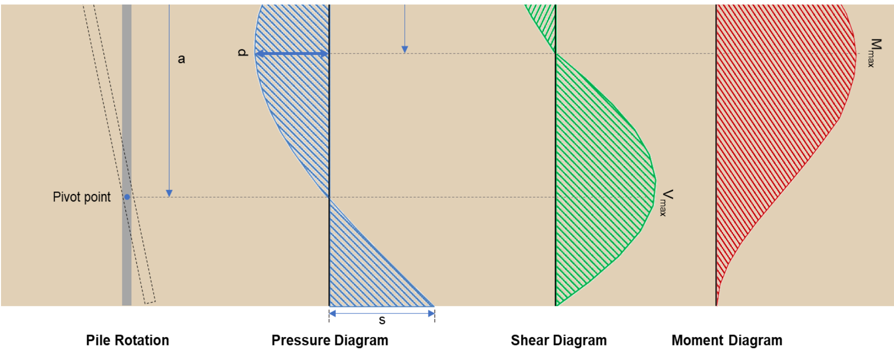
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.84055\ \mathrm{kip/ft^2})}{(0.9\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.93395$$

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.469\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.23392\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.444\ \text{kipft}) + ((-1.469\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 5.007\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.007\ \text{kipft/ft})}{(-0.23392\ \text{kip/ft})}$$

$$E = 21.405\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.007\ \text{kipft/ft}) \times (6\ \text{ft})) + \left(3 \times (-0.23392\ \text{kip/ft}) \times (6\ \text{ft})^2\right)}{(6 \times (5.007\ \text{kipft/ft})) + (4 \times (-0.23392\ \text{kip/ft}) \times (6\ \text{ft}))}$$

$$a = 4.0787\ \text{ft}$$

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

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

$$V_{max} = ((-0.23392\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (21.405\ \text{ft})}{(6\ \text{ft})} + 3\right) \times \left(\frac{(4.0787\ \text{ft})}{(6\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (21.405\ \text{ft})}{(6\ \text{ft})} + 2\right) \times \left(\frac{(4.0787\ \text{ft})}{(6\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 6.5316\ \text{kip}$$

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

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

$$M_{max} = ((-0.23392\ \text{kip/ft}) \times (48\ \text{in}) \times (6\ \text{ft})) \times \left[\left(\frac{(21.405\ \text{ft})}{(6\ \text{ft})} + \frac{(4.0787\ \text{ft})}{2 \times (6\ \text{ft})}\right) - \left[\left(\frac{4 \times (21.405\ \text{ft})}{(6\ \text{ft})} + 3\right) \times \left(\frac{(4.0787\ \text{ft})}{2 \times (6\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (21.405\ \text{ft})}{(6\ \text{ft})} + 2\right) \times \left(\frac{(4.0787\ \text{ft})}{2 \times (6\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 19.081 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(11.984 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

A_{min} - Governing minimum reinforcement area,

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

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

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2 Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

	Ties: **#3(0.375 in) - 10 in**	

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(11.984\text{ kip})}{(2675.2\text{ kip})}$$

$$Ratio = 0.0044797$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2 d - Effective depth

$$d = 0.80\, D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 11.984\text{ kip} \rightarrow 11984\text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(11984\text{ lbf})}{6 \times (2304\text{ in}^2)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$

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

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $V_{s,a}$ - 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 $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((120.08\ \text{kip}) + (50.894\ \text{kip}))$$ $$\phi V_n = 111.14\ \text{kip}$$ **Considering x-direction:** $V_{max} = 6.5316\ \text{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(6.5316\ \text{kip})}{(111.14\ \text{kip})}$$ $$Ratio = 0.058772$$	Status: **PASS** Ratio: **0.060**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \text{in}) \times (48\ \text{in})^2}{6}$$ $$S_m = 18432\ \text{in}^3$$ $\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f'_c)} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
14.5.2.1b	$\phi M_{n,2}$ $$\phi M_{n,2} = \phi\, 0.85\, f'_c\, S_m$$	

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

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

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

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:

$M_{max} = 19.081\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(19.081\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.076446$$

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