Your Project Calculations

Project Name: AP Irrigation TOP-40-TALL-RevA-JB

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=AP%20Irrigation%20TOP-40-TALL-RevA-JB&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/2_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=xY80bWzJOK3yuB5RrC1qesRccI33VFtdScoZiLShEagQckAHe4SpAZc3AkEX7dm7

Array Specification

Product:	Beam
Unique ID:	3P-17-10TOP-SD-24-L-5Hx8W-A2H8
Duty Classification:	SD
Module Width:	44.60 in
Module Length:	67.80in
Number of Rows:	5
Number of Columns:	8
Total Number of Modules:	40
Desired Tilt Angle:	50
Front Edge Clearance:	7
Total Array Height at Tilt:	21.32 ft
Total Frame Length:	45.50 ft
Frame Weight:	3000 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	45.87 ft
Rail Length:	225.50 in
Rail Spacing:	2.82 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	14.20 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 7.00 ft Pile 3: 6.75 ft
Foundation Volume:	12.148 y^3
Foundation Result:	PASSED
Mount Twist:	1.265284 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Carmel Valley, CA 93924, USA
Wind Speed:	87 mph
Snow Load:	0 psf
Design Uplift Pressure:	0.016681 ksf
Design Downforce Pressure:	-0.016681 ksf
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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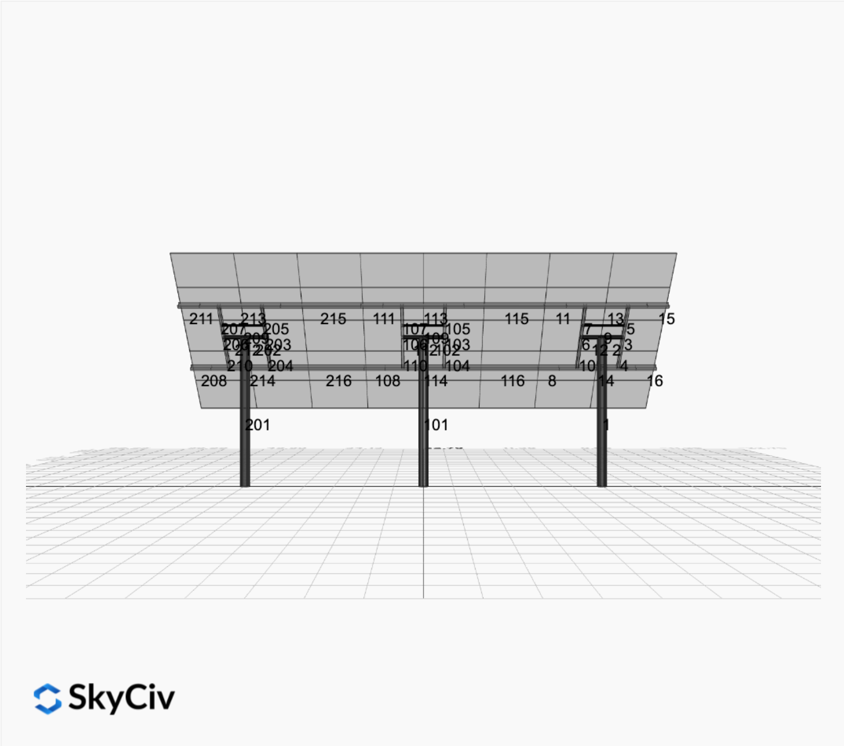
Design Notes:

- 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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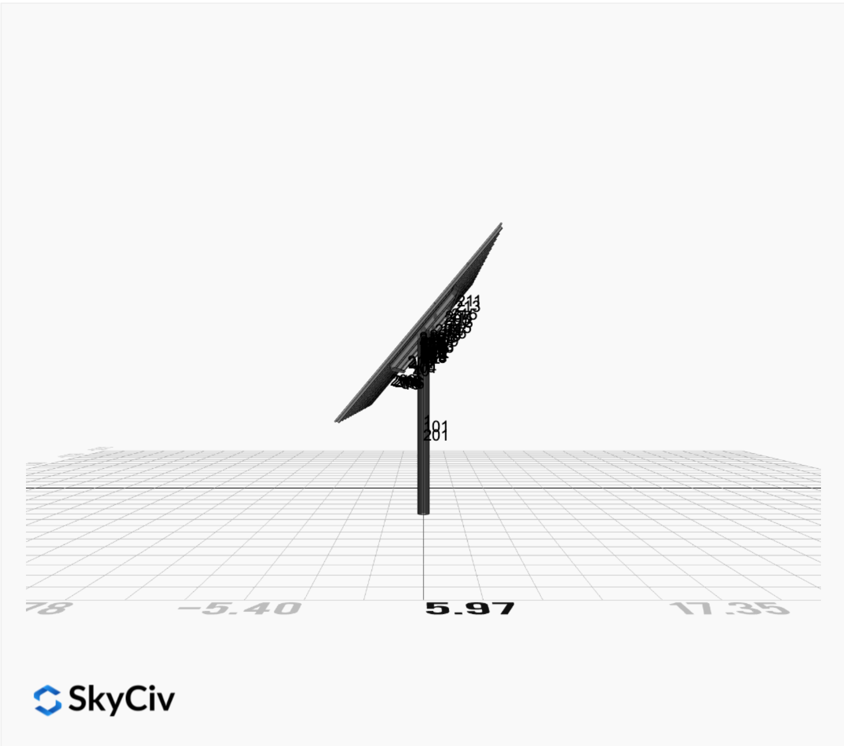
SkyCiv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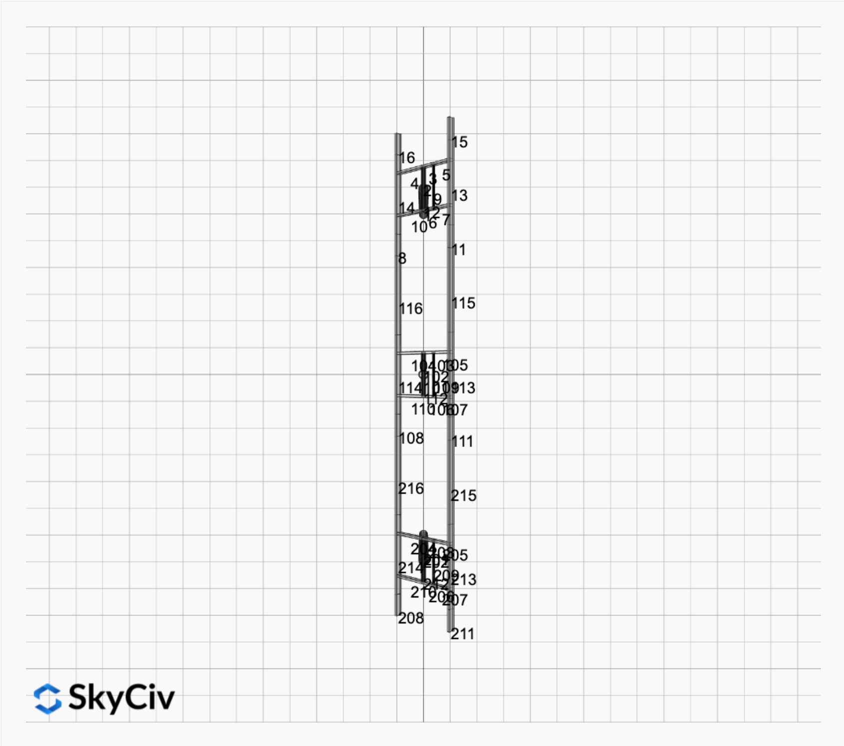

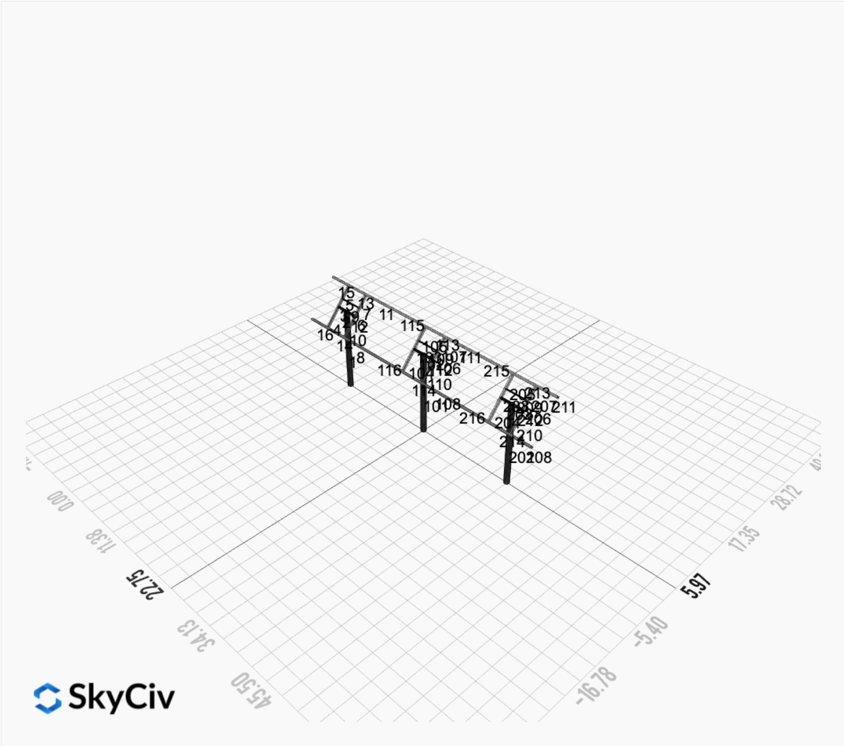

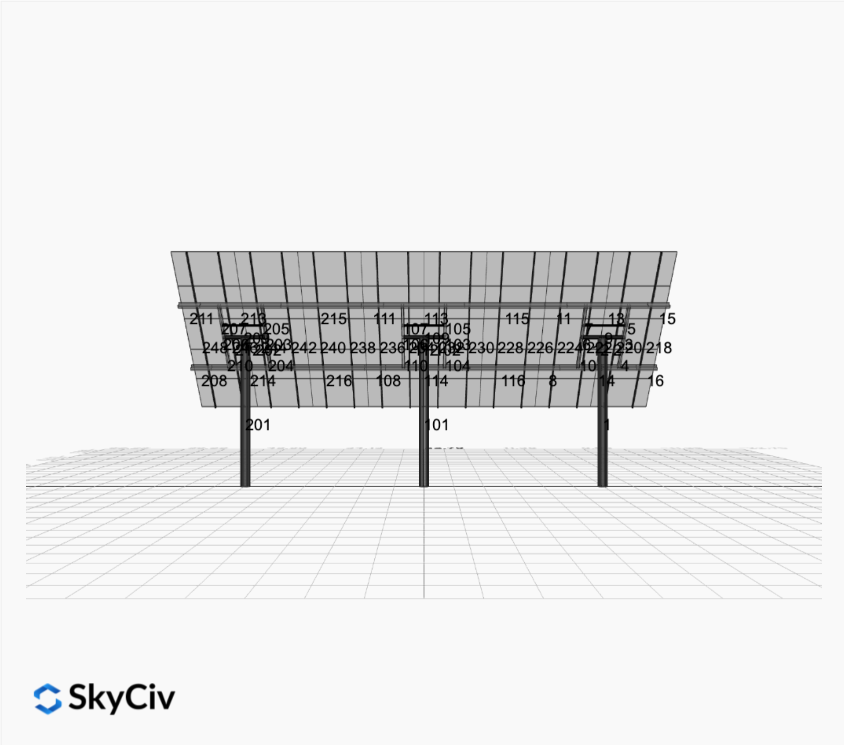
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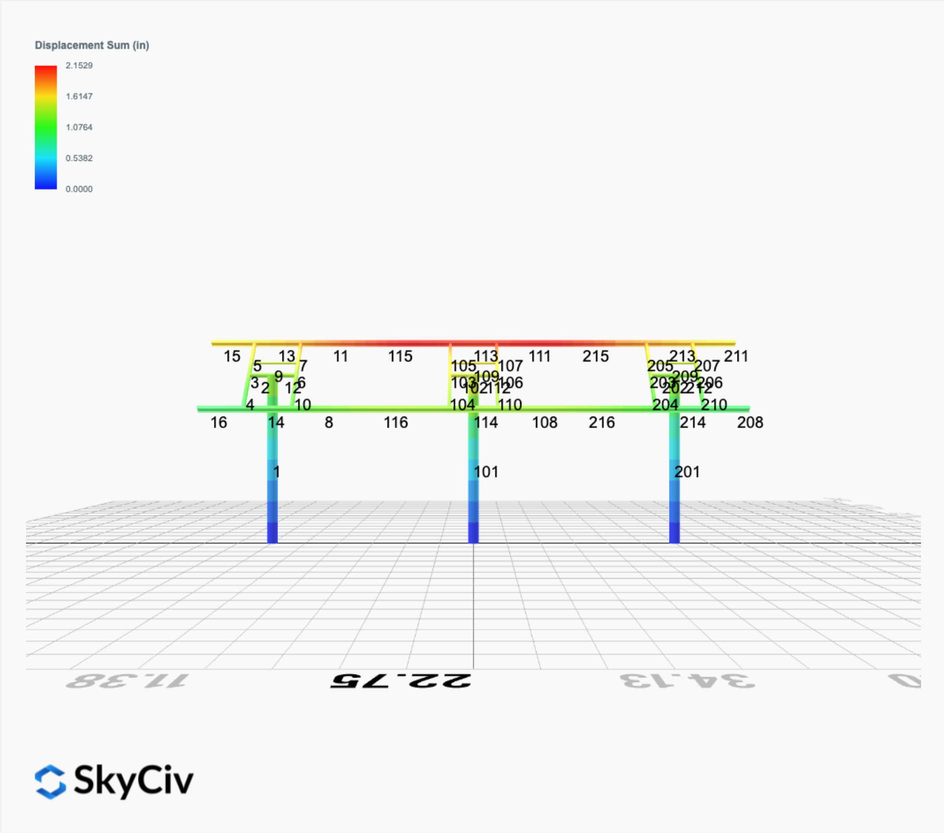
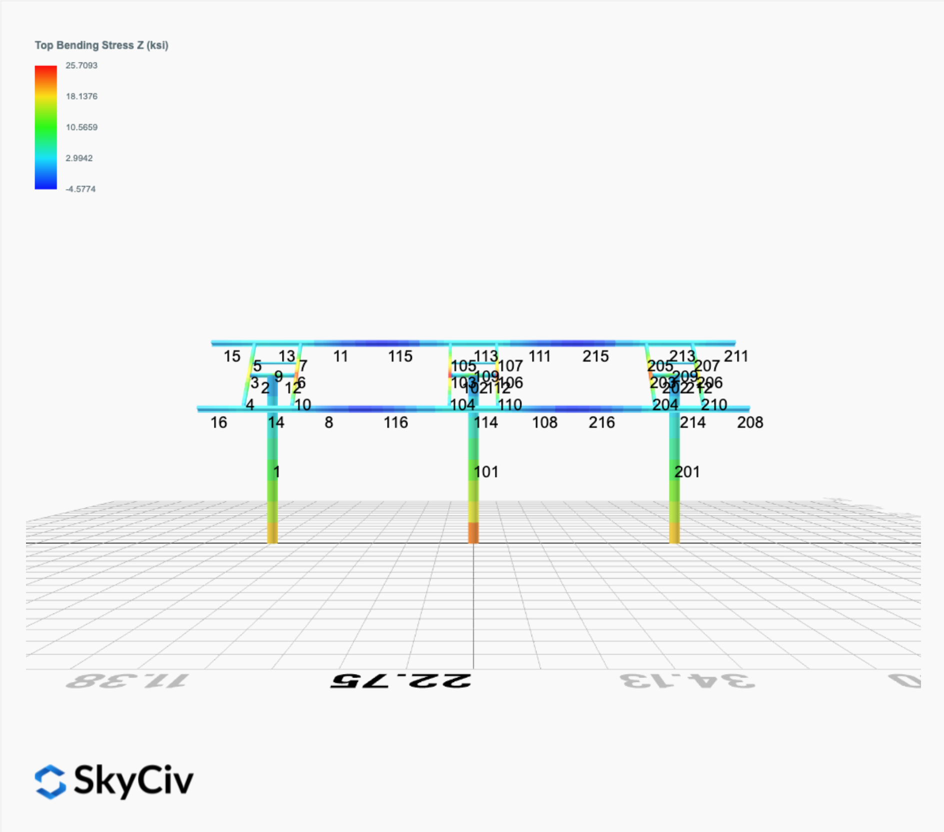
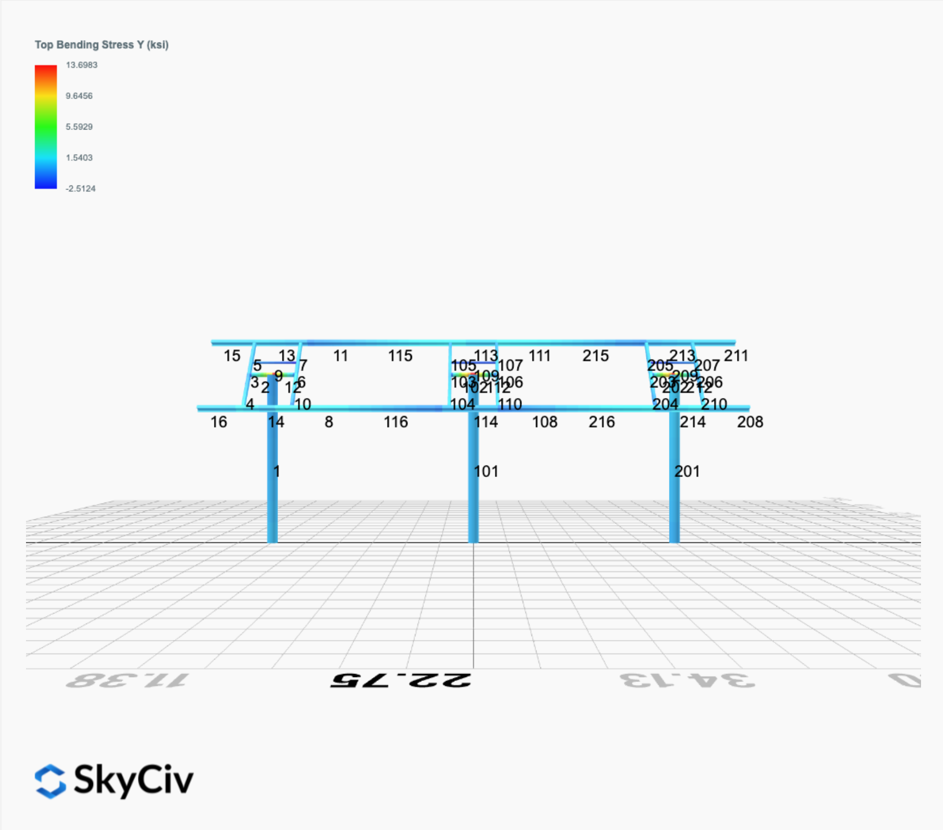
SkyCiv

FEM Results (Envelope Worst Case for each member)



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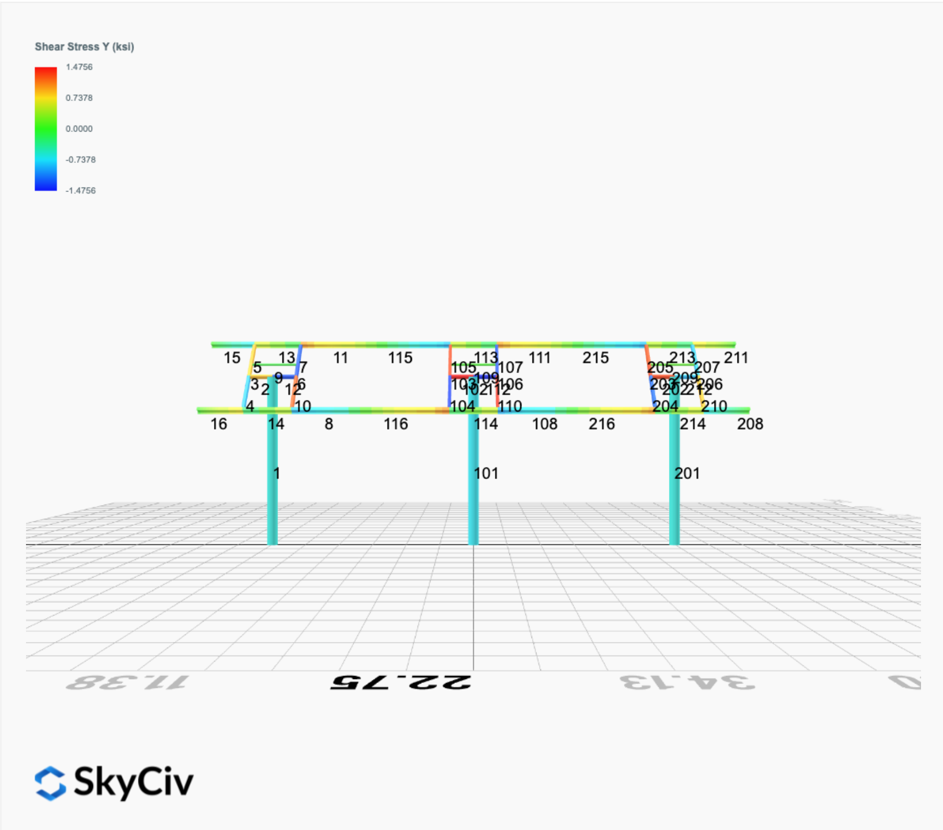
1.5403
-2.5124


Shear Stress Y (ksi)
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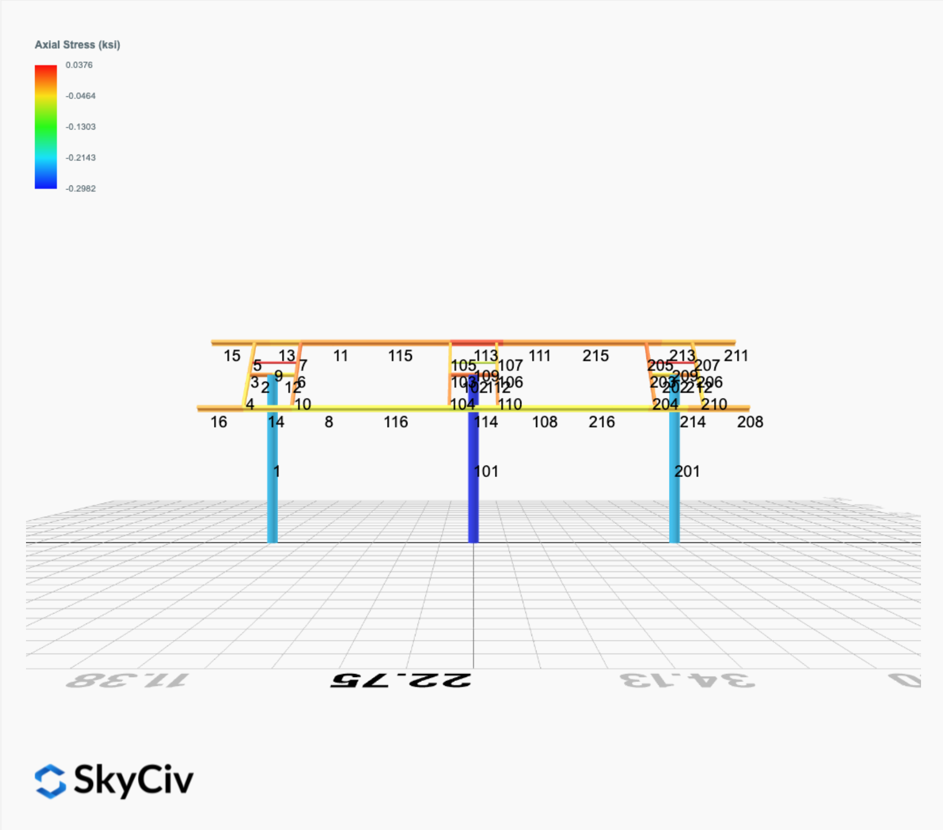
-1.4756


Axial Stress (ksi)
0.0376
-0.0464
-0.1303
-0.2143
-0.2982

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	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 2. D + L	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 3. D + (S or Lr or R)	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 3. D + (S or Lr or R)	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 5b. D + 0.7E	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 8. 0.6D + 0.7E	0.0079	1.3194	0.0255	0.1038	-0.0338	-0.0960
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0611	3.9067	0.1237	0.4714	-0.7726	29.5660
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0860	0.4919	-0.0378	-0.1219	0.6528	-29.5135
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5426	3.4798	0.1034	0.3968	-0.5936	22.1345
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5678	0.9187	-0.0178	-0.0482	0.4755	-22.1751
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5426	3.4798	0.1034	0.3968	-0.5936	22.1345
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5678	0.9187	-0.0178	-0.0482	0.4755	-22.1751
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0131	2.1990	0.0425	0.1731	-0.0564	-0.1599
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0664	3.0271	0.1067	0.4021	-0.7501	29.6300
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0079	1.3194	0.0255	0.1038	-0.0338	-0.0960
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0807	-0.3877	-0.0549	-0.1912	0.6753	-29.4496
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0079	1.3194	0.0255	0.1038	-0.0338	-0.0960

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	5.4852
Shear X	-3.4714
Shear Z	0.1869
Moment X	0.7068
Moment Y (Twist)	1.2652
Moment Z	49.6682

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	3.9067
Shear X	-2.0860
Shear Z	0.1237
Moment X	0.4714
Moment Y (Twist)	0.7726
Moment Z	29.6300

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	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 2. D + L	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 3. D + (S or Lr or R)	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 3. D + (S or Lr or R)	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 5b. D + 0.7E	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 8. 0.6D + 0.7E	-0.0157	1.5179	0.0000	0.0000	0.0000	0.2221
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.4859	4.6595	0.0000	0.0000	0.0000	35.3489
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.4363	0.3992	0.0000	0.0000	0.0000	-34.1434
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8710	4.1271	0.0000	0.0000	0.0000	26.6042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8206	0.9319	0.0000	0.0000	0.0000	-25.5150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.8710	4.1271	0.0000	0.0000	0.0000	26.6042
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8206	0.9319	0.0000	0.0000	0.0000	-25.5150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0262	2.5299	0.0000	0.0000	0.0000	0.3701
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.4754	3.6475	0.0000	0.0000	0.0000	35.2009
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0157	1.5179	0.0000	0.0000	0.0000	0.2221
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.4468	-0.6128	0.0000	0.0000	0.0000	-34.2915
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0157	1.5179	0.0000	0.0000	0.0000	0.2221

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundatino Module. Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	6.5847
Shear X	-4.1290
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0001
Moment Z	59.1262

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	4.6595
Shear X	-2.4859
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	35.3489

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	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 2. D + L	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 3. D + (S or Lr or R)	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 3. D + (S or Lr or R)	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 5b. D + 0.7E	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 8. 0.6D + 0.7E	0.0079	1.3194	-0.0255	-0.1038	0.0338	-0.0960
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0611	3.9067	-0.1237	-0.4714	0.7727	29.5660
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0860	0.4919	0.0378	0.1219	-0.6528	-29.5135
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5426	3.4798	-0.1034	-0.3968	0.5936	22.1345
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5678	0.9187	0.0178	0.0482	-0.4755	-22.1751
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5426	3.4798	-0.1034	-0.3968	0.5936	22.1345
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5678	0.9187	0.0178	0.0482	-0.4755	-22.1751
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0131	2.1990	-0.0425	-0.1731	0.0564	-0.1599
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.0664	3.0271	-0.1067	-0.4021	0.7501	29.6300
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0079	1.3194	-0.0255	-0.1038	0.0338	-0.0960
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0807	-0.3877	0.0549	0.1911	-0.6753	-29.4496
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0079	1.3194	-0.0255	-0.1038	0.0338	-0.0960

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	5.4852
Shear X	-3.4714
Shear Z	-0.1869
Moment X	-0.7067
Moment Y (Twist)	1.2653
Moment Z	49.6687

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	3.9067
Shear X	-2.0860
Shear Z	-0.1237
Moment X	-0.4714
Moment Y (Twist)	0.7727
Moment Z	29.6300

Project Details

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

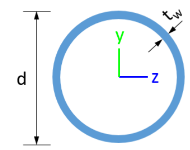
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)					
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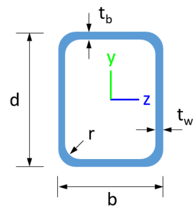
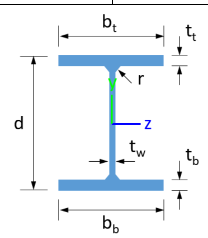
4	4in Pipe Sch 40	4.50	0.24					
11	10in Pipe Sch 40	10.75	0.36					



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



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

Section Properties								
ID	Name	A (in^2)	J (in^4)	I_{yp} (in^4)	I_{zp} (in^4)	I_w (in^6)	S_{yp} (in^3)	S_{zp} (in^3)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38

15	HSS5x3x1/8	1.77	6.02	2.75	6.03	0.51	2.07	2.93
18	W6x9	2.68	0.04	2.20	16.40	17.70	1.72	6.23

Member Properties								
Member ID	Section ID	K_zL (ft)	K_yL (ft)	L_b(ft)	C_b	LST	LSC	LD
1	11	29.82	29.82	14.20	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.17,1.18	300	200	1
4	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
7	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
8	18	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.35,1.34,1.35,1.34,1.35,1.34,1.35,1.34,1.34,1.34,1.35,1.34,1.34,1.34,1.35,1.34,1.35,1.34,1.35,1.34	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
11	18	1.33	1.33	2.05	1.36,1.36,1.36,1.36,1.36,1.36,1.39,1.36,1.41,1.36,1.39,1.36,1.41,1.36,1.38,1.36,1.44,1.36,1.38,1.36,1.44,1.36,1.39,1.36,1.41,1.36	300	200	1
12	4	1.30	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.14,1.16,1.13,1.16,1.14,1.16,1.13,1.16,1.14,1.16,1.12,1.16,1.14,1.16,1.12,1.16,1.14,1.16,1.13,1.16	300	200	1
14	18	4.88	4.00	7.50	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.12,1.15,1.14,1.15,1.12,1.15,1.14,1.15,1.14,1.15	300	200	1
15	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
16	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
101	11	29.82	29.82	14.20	-	300	200	1
102	4	1.30	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
105	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1
106	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18	300	200	1
107	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67	300	200	1

108	18	1.33	1.33	2.05	1.61,1.61,1.61,1.61,1.61,1.61,1.52,1.61,1.48,1.61,1.52,1.61,1.48,1.61,1.54,1.61,1.42,1.61,1.54,1.61,1.42,1.61,1.52,1.61,1.49,1.61	300	200	1
109	1	2.60	2.60	4.00	-	300	200	1
110	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
111	18	1.33	1.33	2.05	1.55,1.55,1.55,1.55,1.55,1.55,1.39,1.55,1.32,1.55,1.39,1.55,1.32,1.55,1.41,1.55,1.25,1.55,1.41,1.55,1.25,1.55,1.38,1.55,1.33,1.55	300	200	1
112	4	1.30	1.30	2.00	-	300	200	1
113	18	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.06,1.09,1.06,1.08,1.06,1.09,1.06,1.07,1.06,1.13,1.06,1.07,1.06,1.13,1.06,1.08,1.06,1.09,1.06	300	200	1
114	18	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.05,1.06,1.05,1.06,1.05,1.06,1.05,1.06,1.05,1.07,1.05,1.06,1.05,1.07,1.05,1.06,1.05,1.06,1.05	300	200	1
115	18	4.84	4.84	7.45	1.13,1.13,1.13,1.13,1.13,1.13,1.10,1.13,1.09,1.13,1.10,1.13,1.09,1.13,1.11,1.13,1.07,1.13,1.11,1.13,1.07,1.13,1.10,1.13,1.09,1.13	300	200	1
116	18	4.84	4.84	7.45	1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.14,1.12,1.14,1.13,1.14,1.12,1.14,1.13,1.14,1.11,1.14,1.13,1.14,1.11,1.14,1.13,1.14,1.12,1.14	300	200	1
201	11	29.82	29.82	14.20	-	300	200	1
202	4	1.30	1.30	2.00	-	300	200	1
203	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19	300	200	1
204	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
205	15	1.52	1.52	2.33	1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.66,1.67,1.67,1.67,1.67,1.67	300	200	1
206	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.17,1.18	300	200	1
207	15	1.52	1.52	2.33	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
208	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
209	1	2.60	2.60	4.00	-	300	200	1
210	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	300	200	1
211	18	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33	300	200	1
212	4	1.30	1.30	2.00	-	300	200	1
213	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.14,1.16,1.13,1.16,1.14,1.16,1.13,1.16,1.14,1.16,1.12,1.16,1.14,1.16,1.12,1.16,1.14,1.16,1.13,1.16	300	200	1
214	18	4.88	4.00	7.50	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.12,1.15,1.14,1.15,1.12,1.15,1.14,1.15,1.14,1.15	300	200	1
215	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.11,1.09,1.10,1.09,1.11,1.09,1.10,1.09,1.12,1.09,1.10,1.09,1.12,1.09,1.10,1.09,1.11,1.09	300	200	1
216	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.09,1.09	300	200	1

Member Design Capacity

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	535.87	267.86	147.68	147.68	160.76	160.76
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.67	6.45	30.09	45.74
14	120.60	98.23	19.76	6.45	30.09	45.74
15	120.60	96.18	23.36	6.45	30.09	45.74
16	120.60	96.18	23.36	6.45	30.09	45.74
101	535.87	267.86	147.68	147.68	160.76	160.76
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.64	6.45	30.09	45.74
114	120.60	98.23	18.56	6.45	30.09	45.74
115	120.60	89.27	18.94	6.45	30.09	45.74
116	120.60	89.27	19.64	6.45	30.09	45.74
201	535.87	267.86	147.68	147.68	160.76	160.76
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	96.18	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	96.18	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	19.66	6.45	30.09	45.74
214	120.60	98.23	19.77	6.45	30.09	45.74
215	120.60	89.27	19.33	6.45	30.09	45.74
216	120.60	89.27	19.23	6.45	30.09	45.74

Design Ratio

Member ID	P	M_z	M_y	V_y	V_z	(P,M_z,M_y)	Worst LC	KL/r	δ	Status
1	0.020	0.336	0.013	0.022	0.001	0.351	#13	0.487	Not Required	Pass
2	0.000	0.184	0.165	0.046	0.035	0.349	#13	0.052	Not Required	Pass
3	0.005	0.415	0.048	0.041	0.008	0.457	#13	0.044	Not Required	Pass
4	0.004	0.412	0.075	0.042	0.011	0.489	#13	0.078	Not Required	Pass
5	0.004	0.257	0.053	0.042	0.007	0.260	#13	0.073	Not Required	Pass
6	0.007	0.561	0.093	0.058	0.016	0.646	#13	0.044	Not Required	Pass
7	0.007	0.348	0.115	0.056	0.016	0.367	#13	0.073	Not Required	Pass
8	0.002	0.083	0.042	0.030	0.006	0.125	#13	0.088	Not Required	Pass
9	0.001	0.041	0.061	0.003	0.002	0.103	#13	0.198	Not Required	Pass
10	0.007	0.543	0.132	0.055	0.018	0.579	#13	0.078	Not Required	Pass
11	0.001	0.082	0.040	0.031	0.006	0.122	#13	0.088	Not Required	Pass
12	0.001	0.305	0.243	0.066	0.048	0.548	#13	0.052	Not Required	Pass
13	0.002	0.089	0.117	0.043	0.008	0.167	#15	0.265	Not Required	Pass
14	0.002	0.087	0.119	0.041	0.008	0.160	#15	0.177	Not Required	Pass
15	0.000	0.017	0.017	0.013	0.002	0.032	#13	Not Required	Not Required	Pass
16	0.000	0.017	0.017	0.013	0.002	0.032	#13	Not Required	Not Required	Pass
101	0.025	0.400	0.000	0.026	0.000	0.413	#13	0.487	Not Required	Pass
102	0.001	0.299	0.247	0.069	0.048	0.547	#13	0.052	Not Required	Pass
103	0.007	0.582	0.079	0.059	0.012	0.650	#13	0.044	Not Required	Pass
104	0.006	0.596	0.120	0.060	0.015	0.686	#13	0.078	Not Required	Pass
105	0.007	0.361	0.123	0.058	0.018	0.389	#13	0.073	Not Required	Pass
106	0.007	0.582	0.079	0.059	0.012	0.650	#13	0.044	Not Required	Pass
107	0.007	0.361	0.123	0.058	0.018	0.389	#13	0.073	Not Required	Pass
108	0.002	0.072	0.043	0.033	0.006	0.094	#13	0.088	Not Required	Pass
109	0.004	0.034	0.046	0.001	0.000	0.081	#13	0.198	Not Required	Pass
110	0.006	0.596	0.120	0.060	0.015	0.686	#13	0.078	Not Required	Pass
111	0.001	0.082	0.043	0.031	0.006	0.104	#13	0.088	Not Required	Pass
112	0.001	0.299	0.247	0.069	0.048	0.547	#13	0.052	Not Required	Pass
113	0.002	0.099	0.122	0.043	0.008	0.205	#13	0.265	Not Required	Pass
114	0.003	0.124	0.122	0.044	0.008	0.229	#13	0.265	Not Required	Pass
115	0.002	0.132	0.068	0.031	0.006	0.193	#13	0.321	Not Required	Pass
116	0.002	0.125	0.068	0.033	0.006	0.186	#13	0.321	Not Required	Pass
201	0.020	0.336	0.013	0.022	0.001	0.351	#13	0.487	Not Required	Pass
202	0.001	0.305	0.243	0.066	0.048	0.548	#13	0.052	Not Required	Pass
203	0.007	0.561	0.093	0.058	0.016	0.646	#13	0.044	Not Required	Pass
204	0.007	0.543	0.132	0.055	0.018	0.579	#13	0.078	Not Required	Pass
205	0.007	0.348	0.115	0.056	0.016	0.367	#13	0.073	Not Required	Pass
206	0.005	0.415	0.048	0.041	0.008	0.457	#13	0.044	Not Required	Pass
207	0.004	0.257	0.053	0.042	0.007	0.260	#13	0.073	Not Required	Pass
208	0.000	0.017	0.017	0.013	0.002	0.032	#13	Not Required	Not Required	Pass
209	0.001	0.041	0.061	0.003	0.002	0.103	#13	0.198	Not Required	Pass
210	0.004	0.412	0.075	0.042	0.011	0.489	#13	0.078	Not Required	Pass
211	0.000	0.017	0.017	0.013	0.002	0.032	#13	Not Required	Not Required	Pass
212	0.000	0.184	0.165	0.046	0.035	0.349	#13	0.052	Not Required	Pass
213	0.002	0.089	0.117	0.043	0.008	0.167	#15	0.177	Not Required	Pass
214	0.002	0.087	0.119	0.041	0.008	0.160	#15	0.265	Not Required	Pass
215	0.002	0.136	0.068	0.031	0.006	0.193	#13	0.321	Not Required	Pass
216	0.002	0.131	0.069	0.030	0.006	0.191	#13	0.321	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

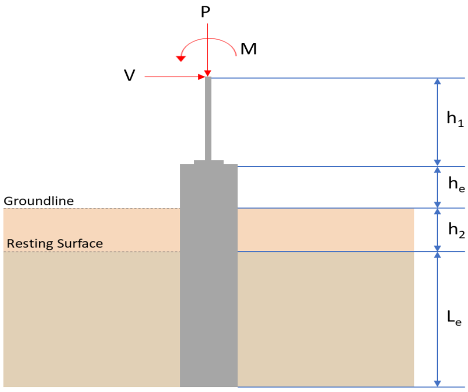
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - 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)	3.907	5.485
V_x (kip)	-2.086	-3.471
V_z (kip)	0.124	0.187
M_x (kipft)	0.471	0.707
M_z (kipft)	29.630	49.668

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,

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

$$H_o = \frac{(-2.086\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.33217\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.63 \text{ kipft}) + ((-2.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.7182 \text{ kipft/ft}$$

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:
$L_{e,x} = 6.3132 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.124 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.019745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.471 \text{ kipft}) + ((0.124 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.075 \text{ kipft/ft}$$

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:
$L_{e,z} = 2.0336 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.3132 \text{ ft}), (2.0336 \text{ ft})]$$

$$L_{e,req} = 6.313 \text{ ft}$$

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

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

$$Ratio = \frac{(6.313 \text{ ft})}{(6.75 \text{ ft})}$$

$$Ratio = 0.93526$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\, D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(3.907 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.24419 \text{ kip/ft}^2$$

$$q = 0.24419 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.24419 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.12209$$

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

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:

$H_o = -0.33217 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.7182 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7182 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (4.7182 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6353 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.7182 \text{ kipft/ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.7182 \text{ kipft/ft})) + (2 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.25113 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.7182 \text{ kipft/ft})) + ((-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.94738 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6353 \text{ ft})}{2}$$

$$p_a = 0.34765 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.25113 \text{ kip/ft}^2)}{(0.34765 \text{ kip/ft}^2)}$$

$$Ratio = 0.72237$$

Status: **PASS**
Ratio: **0.720**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.94738\ \text{kip/ft}^2)}{(1.0125\ \text{kip/ft}^2)}$$

$$Ratio = 0.93569$$

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

Considering z-direction:

$H_o = 0.019745\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.075\ \text{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (0.019745\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (0.075\ \text{kipft/ft})) + (4 \times (0.019745\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.805\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.075\ \text{kipft/ft})) + (3 \times (0.019745\ \text{kip/ft}) \times (6.75\ \text{ft}))]^{\ 2}}{(6.75\ \text{ft})^2 \times [(3 \times (0.075\ \text{kipft/ft})) + (2 \times (0.019745\ \text{kip/ft}) \times (6.75\ \text{ft}))]}$$

$$p = 0.016401\ \text{kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.075\ \text{kipft/ft})) + ((0.019745\ \text{kip/ft}) \times (6.75\ \text{ft}))]}{(6.75\ \text{ft})^2}$$

$$s = 0.037304\ \text{kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(4.805\ \text{ft})}{2}$$

$$p_a = 0.36038\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.016401\ \text{kip/ft}^2)}{(0.36038\ \text{kip/ft}^2)}$$

$$Ratio = 0.045511$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

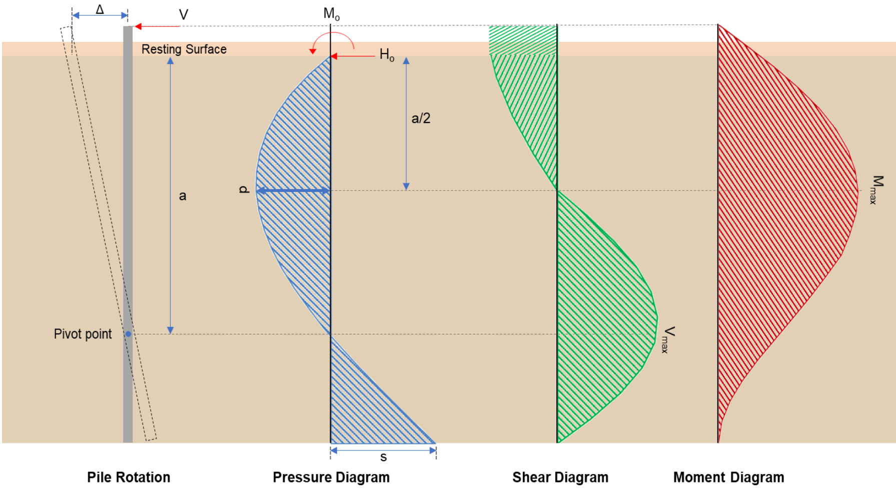
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.037304\ \text{kip/ft}^2)}{(1.0125\ \text{kip/ft}^2)}$$

$$Ratio = 0.036844$$

Status: **PASS**
Ratio: **0.040**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.471\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.55271\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.668\ \text{kipft}) + ((-3.471\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 7.9089\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.9089\ \text{kipft/ft})}{(-0.55271\ \text{kip/ft})}$$

$$E = 14.309\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9089\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (-0.55271\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (7.9089\ \text{kipft/ft})) + (4 \times (-0.55271\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.6346\ \text{ft}$$

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

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

$$V_{max} = ((-0.55271\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.309\ \text{ft})}{(6.75\ \text{ft})} + 3\right) \times \left(\frac{(4.6346\ \text{ft})}{(6.75\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (14.309\ \text{ft})}{(6.75\ \text{ft})} + 2\right) \times \left(\frac{(4.6346\ \text{ft})}{(6.75\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 9.7537\ \text{kip}$$

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

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

$$M_{max} = ((-0.55271\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(14.309\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.6346\ \text{ft})}{2 \times (6.75\ \text{ft})} \right) - \left[\left(\frac{4 \times (14.309\ \text{ft})}{(6.75\ \text{ft})} + 3 \right) \times \left(\frac{(4.6346\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (14.309\ \text{ft})}{(6.75\ \text{ft})} + 2 \right) \times \left(\frac{(4.6346\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^4 \right] \right]$$

$$M_{max} = 31.56\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.707\ \text{kipft}) + ((0.187\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.11258\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11258\ \text{kipft/ft})}{(0.029777\ \text{kip/ft})}$$

$$E = 3.7807\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11258\ \text{kipft/ft}) \times (6.75\ \text{ft})) + \left(3 \times (0.029777\ \text{kip/ft}) \times (6.75\ \text{ft})^2\right)}{(6 \times (0.11258\ \text{kipft/ft})) + (4 \times (0.029777\ \text{kip/ft}) \times (6.75\ \text{ft}))}$$

$$a = 4.8057\ \text{ft}$$

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

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

$$V_{max} = ((0.029777\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7807\ \text{ft})}{(6.75\ \text{ft})} + 3 \right) \times \left(\frac{(4.8057\ \text{ft})}{(6.75\ \text{ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.7807\ \text{ft})}{(6.75\ \text{ft})} + 2 \right) \times \left(\frac{(4.8057\ \text{ft})}{(6.75\ \text{ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.19727\ \text{kip}$$

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

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

$$M_{max} = ((0.029777\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(3.7807\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.8057\ \text{ft})}{2 \times (6.75\ \text{ft})} \right) - \left[\left(\frac{4 \times (3.7807\ \text{ft})}{(6.75\ \text{ft})} + 3 \right) \times \left(\frac{(4.8057\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7807\ \text{ft})}{(6.75\ \text{ft})} + 2 \right) \times \left(\frac{(4.8057\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.59398 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(5.485 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times (2304 \text{ in}^2)\right)\right]$$

$$A_{st,required} = -84.414 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.414 \text{ in}^2\right), \left(0.0018 \times (2304 \text{ in}^2)\right)\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

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 $$\phi P_N = \phi\, 0.80\, \left[\left(0.85\, f'_{ck}\, \left[A_g - A_{st}\right]\right) + \left(f_{yk}\, A_{st}\right)\right]$$ $$\phi P_N = (0.65) \times 0.80 \times \left[\left(0.85 \times (2.5\ \mathrm{ksi}) \times \left[\left(2304\ \mathrm{in}^2\right) - \left(4.2951\ \mathrm{in}^2\right)\right]\right) + \left((60\ \mathrm{ksi}) \times \left(4.2951\ \mathrm{in}^2\right)\right)\right]$$ $$\phi P_N = 2675.2\ \mathrm{kip}$$ $Ratio$ - Capacity $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(5.485\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$ $$Ratio = 0.0020503$$	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:** $b_w = 48\ \mathrm{in}$ - Effective width, d - Effective depth $$d = 0.80\, D$$ $$d = 0.80 \times (48\ \mathrm{in})$$ $$d = 38.4\ \mathrm{in}$$ λ_s - size effect modification factor $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1\right]$$ $$\lambda_s = MIN\left[\sqrt{\frac{2}{1 + \frac{(38.4\ \mathrm{in})}{10}}}, 1\right]$$ $$\lambda_s = 0.64282$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $V_{c,max}$ - Max shear strength of concrete $$V_{c,max} = 5\, \lambda_s\, \sqrt{f'_{ck}}\, b_w\, d$$ $$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,max} = 296.21\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 5.485\ \mathrm{kip} \rightarrow 5485\ \mathrm{lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right]\, b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(5485\ \mathrm{lbf})}{6 \times (2304\ \mathrm{in}^2)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{c,a} = 119.22\ \mathrm{kip}$$ The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $V_{c,b}$ - 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 $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\ \mathrm{kip}), (119.22\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$ $$V_c = 119.22\ \mathrm{kip}$$	

22.5.1.2	The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $V_{s,a}$ - Shear strength of steel (a) $$V_{s,a} = 8\sqrt{f'_{ck}}\, b_w\, d$$ $$V_{s,a} = 8 \times \sqrt{(2500\ \mathrm{psi})} \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$ $$V_{s,a} = 737.28\ \mathrm{kip}$$ 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 $$\phi V_n = \phi\ (V_c + V_s)$$ $$\phi V_n = (0.65) \times ((119.22\ \mathrm{kip}) + (50.894\ \mathrm{kip}))$$ $$\phi V_n = 110.57\ \mathrm{kip}$$ **Considering x-direction:** $V_{max} = 9.7537\ \mathrm{kip}$ - Maximum shear force in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(9.7537\ \mathrm{kip})}{(110.57\ \mathrm{kip})}$$ $$Ratio = 0.088212$$	Status: **PASS** Ratio: **0.090**
	Considering z-direction: $V_{max} = 0.19727\ \mathrm{kip}$ - Maximum shear force in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{V_{max}}{\phi V_n}$$ $$Ratio = \frac{(0.19727\ \mathrm{kip})}{(110.57\ \mathrm{kip})}$$ $$Ratio = 0.0017841$$	Status: **PASS** Ratio: **0.000**
	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\, D^2}{6}$$ $$S_m = \frac{(48\ \mathrm{in}) \times (48\ \mathrm{in})^2}{6}$$ $$S_m = 18432\ \mathrm{in}^3$$	

	$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$ $$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{(f_c')} \times S_m$$ $$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5ksi)} \times 18432.001in^3$$ $$\phi M_{n,1} = 249.600kipft$$	
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:** $M_{max} = 31.56\ \text{kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(31.56\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.12644$$	Status: **PASS** Ratio: **0.130**
	Considering z-direction: $M_{max} = 0.59398\ \text{kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.59398\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.0023797$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

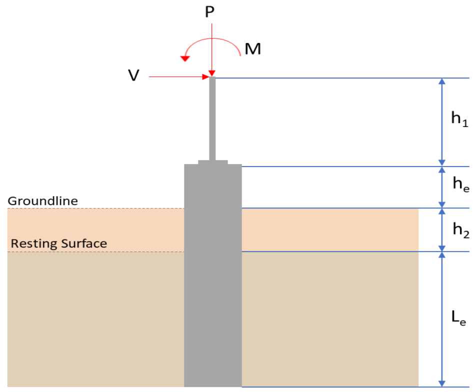
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - 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)	3.907	5.485
V_x (kip)	-2.086	-3.471
V_z (kip)	-0.124	-0.187
M_x (kipft)	-0.471	-0.707
M_z (kipft)	29.630	49.669

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,

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

$$H_o = \frac{(-2.086\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.33217\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.63\ \mathrm{kipft}) + ((-2.086\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 4.7182\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.3132\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.124\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.019745\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.471\ \mathrm{kipft}) + ((-0.124\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 0.075\ \mathrm{kipft/ft}$$

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:
$L_{e,z} = 1.6009\ \mathrm{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.3132\ \mathrm{ft}), (1.6009\ \mathrm{ft})]$$

$$L_{e,req} = 6.313\ \mathrm{ft}$$

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

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

$$Ratio = \frac{(6.313\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.93526$$

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

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

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

$$q = \frac{(3.907\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.24419\ \mathrm{kip/ft}^2$$

$q = 0.24419 \text{ kip/ft}$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.24419 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.12209$$

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

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:

$H_o = -0.33217 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.7182 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7182 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (4.7182 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6353 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.7182 \text{ kipft/ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]^{\,2}}{(6.75 \text{ ft})^2 \times [(3 \times (4.7182 \text{ kipft/ft})) + (2 \times (-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.25113 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.7182 \text{ kipft/ft})) + ((-0.33217 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.94738 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6353 \text{ ft})}{2}$$

$$p_a = 0.34765 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.25113 \text{ kip/ft}^2\right)}{\left(0.34765 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.72237$$

Status: **PASS**
Ratio: **0.720**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.94738 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.93569$$

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

Considering z-direction:

$H_o = -0.019745 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.075 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.075 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.019745 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (0.075 \text{ kipft/ft})) + (4 \times (-0.019745 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.805 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.075 \text{ kipft/ft})) + (3 \times (-0.019745 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.075 \text{ kipft/ft})) + (2 \times (-0.019745 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = -0.0039481 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.075 \text{ kipft/ft})) + ((-0.019745 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.0022018 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.805 \text{ ft})}{2}$$

$$p_a = 0.36038 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(-0.0039481 \text{ kip/ft}^2)}{(0.36038 \text{ kip/ft}^2)}$$

$$Ratio = -0.010955$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

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

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

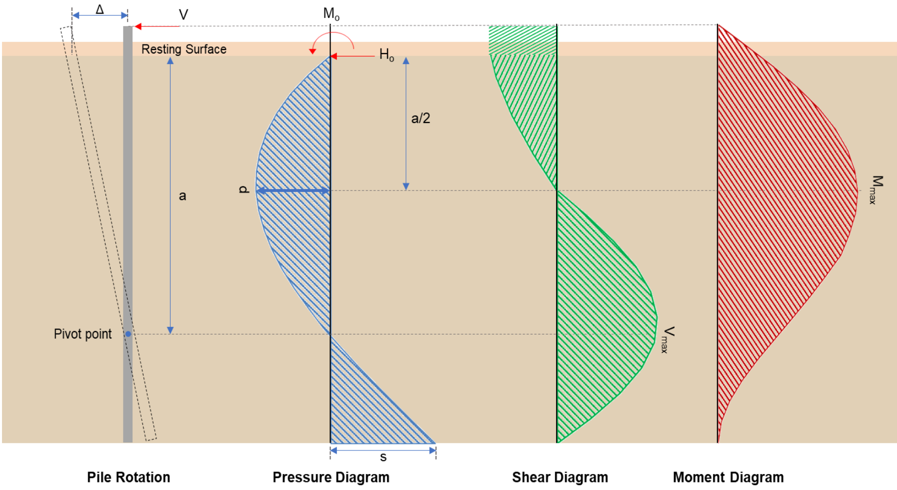
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.0022018 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0021746$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.471 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.55271 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.669 \text{ kipft}) + ((-3.471 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.9091 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.9091 \text{ kipft/ft})}{(-0.55271 \text{ kip/ft})}$$

$$E = 14.31 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.9091 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.55271 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (7.9091 \text{ kipft/ft})) + (4 \times (-0.55271 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6346 \text{ ft}$$

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

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

$$V_{max} = ((-0.55271 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.31 \text{ ft})}{(6.75 \text{ ft})} + 3\right) \times \left(\frac{(4.6346 \text{ ft})}{(6.75 \text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (14.31 \text{ ft})}{(6.75 \text{ ft})} + 2\right) \times \left(\frac{(4.6346 \text{ ft})}{(6.75 \text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 9.7539 \text{ kip}$$

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

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

$$M_{max} = ((-0.55271\ \mathrm{kip/ft})\times(48\ \mathrm{in})\times(6.75\ \mathrm{ft}))\times\left[\left(\frac{(14.31\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+\frac{(4.6346\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4\times(14.31\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+3\right)\times\left(\frac{(4.6346\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^3\right]+\left[\left(\frac{3\times(14.31\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+2\right)\times\left(\frac{(4.6346\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 31.561\ \mathrm{kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.707\ \mathrm{kipft})+((-0.187\ \mathrm{kip})\times(0\ \mathrm{ft}))}{1.57\times(48\ \mathrm{in})}$$

$$M_o = 0.11258\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11258\ \mathrm{kipft/ft})}{(-0.029777\ \mathrm{kip/ft})}$$

$$E = 3.7807\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4\times(0.11258\ \mathrm{kipft/ft})\times(6.75\ \mathrm{ft}))+\left(3\times(-0.029777\ \mathrm{kip/ft})\times(6.75\ \mathrm{ft})^2\right)}{(6\times(0.11258\ \mathrm{kipft/ft}))+(4\times(-0.029777\ \mathrm{kip/ft})\times(6.75\ \mathrm{ft}))}$$

$$a = 4.8057\ \mathrm{ft}$$

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

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

$$V_{max} = ((-0.029777\ \mathrm{kip/ft})\times(48\ \mathrm{in}))\times\left[1-\left[3\times\left(\frac{4\times(3.7807\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+3\right)\times\left(\frac{(4.8057\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^2\right]+\left[4\times\left(\frac{3\times(3.7807\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+2\right)\times\left(\frac{(4.8057\ \mathrm{ft})}{(6.75\ \mathrm{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.19727\ \mathrm{kip}$$

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

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

$$M_{max} = ((-0.029777\ \mathrm{kip/ft})\times(48\ \mathrm{in})\times(6.75\ \mathrm{ft}))\times\left[\left(\frac{(3.7807\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+\frac{(4.8057\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right) - \left[\left(\frac{4\times(3.7807\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+3\right)\times\left(\frac{(4.8057\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^3\right]+\left[\left(\frac{3\times(3.7807\ \mathrm{ft})}{(6.75\ \mathrm{ft})}+2\right)\times\left(\frac{(4.8057\ \mathrm{ft})}{2\times(6.75\ \mathrm{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.59398 \text{ kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(5.485\text{ kip})}{(0.65)\times(0.8)} - \left(0.85\times(2.5\text{ ksi})\times\left(2304\text{ in}^2\right)\right)}{(60\text{ ksi}) - (0.85\times(2.5\text{ ksi}))}, \left(0.08\times\left(2304\text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.414 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.414\text{ in}^2\right), \left(0.0018\times\left(2304\text{ in}^2\right)\right)\right]$$

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

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**	
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 $$Ratio = \frac{P}{\phi P_N}$$ $$Ratio = \frac{(5.485\text{ kip})}{(2675.2\text{ kip})}$$ $$Ratio = 0.0020503$$	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 $f'_{ck} = 2.5\text{ ksi} \rightarrow 2500\text{ psi}$, $P = 5.485\text{ kip} \rightarrow 5485\text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a) $$V_{c,a} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \frac{P}{6\, A_g}\right] b_w\, d$$ $$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + \frac{(5485\text{ lbf})}{6 \times \left(2304\text{ in}^2\right)}\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,a} = 119.22\text{ kip}$$ 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) $$V_{c,b} = \left[2\, \lambda_s\, \sqrt{f'_{ck}} + \left(0.05\, f'_{ck}\right)\right] b_w\, d$$ $$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500\text{ psi})} + (0.05 \times (2500\text{ psi}))\right] \times (48\text{ in}) \times (38.4\text{ in})$$ $$V_{c,b} = 348.89\text{ kip}$$ V_c - Governing shear strength of concrete $$V_c = Min\left[V_{c,max}, V_{c,a}, V_{c,b}\right]$$ $$V_c = Min\left[(296.21\text{ kip}), (119.22\text{ kip}), (348.89\text{ kip})\right]$$ $$V_c = 119.22\text{ kip}$$	

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

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

$$\phi V_n = (0.65) \times ((119.22 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.57 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.7539 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(9.7539 \text{ kip})}{(110.57 \text{ kip})}$$

$$Ratio = 0.088213$$

Status: **PASS**
Ratio: **0.090**

Considering z-direction:

$V_{max} = 0.19727 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.19727 \text{ kip})}{(110.57 \text{ kip})}$$

$$Ratio = 0.0017841$$

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:** $M_{max} = 31.561 \text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(31.561 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.12645$$	Status: **PASS** Ratio: **0.130**
	Considering z-direction: $M_{max} = 0.59398 \text{ kipft}$ - Maximum moment in the z-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(0.59398 \text{ kipft})}{(249.6 \text{ kipft})}$$ $$Ratio = 0.0023797$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

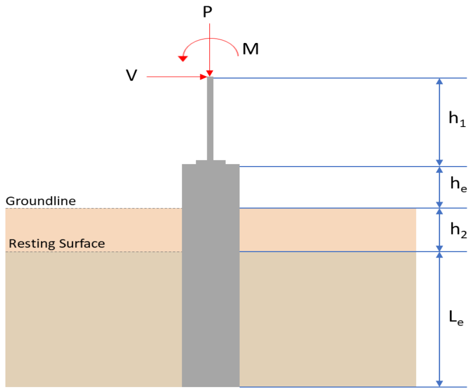
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
$b = 48\ \mathrm{in}$ - Pile width
$D = 48\ \mathrm{in}$ - Pile depth
$L = 7\ \mathrm{ft}$ - 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)	4.660	6.585
V_x (kip)	-2.486	-4.129
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	35.349	59.126

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,

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

$$H_o = \frac{(-2.486\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.39586\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.349\ \mathrm{kipft}) + ((-2.486\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.6288\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.639\ \mathrm{ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(6.639\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 6.639\ \mathrm{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7\ \mathrm{ft}) - (0\ \mathrm{ft}) - (0\ \mathrm{ft})$$

$$L_e = 7\ \mathrm{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(6.639\ \mathrm{ft})}{(7\ \mathrm{ft})}$$

$$Ratio = 0.94843$$

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

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

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

$$q = \frac{(4.66\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.29125\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.29125\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.14563$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.75$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.39586\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 5.6288\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6288\ \mathrm{kipft/ft}) \times (7\ \mathrm{ft})) + \left(3 \times (-0.39586\ \mathrm{kip/ft}) \times (7\ \mathrm{ft})^2\right)}{(6 \times (5.6288\ \mathrm{kipft/ft})) + (4 \times (-0.39586\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))}$$

$$a = 4.8108\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6288\ \mathrm{kipft/ft})) + (3 \times (-0.39586\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]^{\ 2}}{(7\ \mathrm{ft})^2 \times [(3 \times (5.6288\ \mathrm{kipft/ft})) + (2 \times (-0.39586\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}$$

$$p = 0.27214\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (5.6288\ \mathrm{kipft/ft})) + ((-0.39586\ \mathrm{kip/ft}) \times (7\ \mathrm{ft}))]}{(7\ \mathrm{ft})^2}$$

$$s = 1.0392\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.8108\ \mathrm{ft})}{2}$$

$$p_a = 0.36081\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.27214\ \mathrm{kip/ft^2}\right)}{\left(0.36081\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.75425$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (7\ \mathrm{ft})$$

$$p_s = 1.05\ \mathrm{kip/ft^2}$$

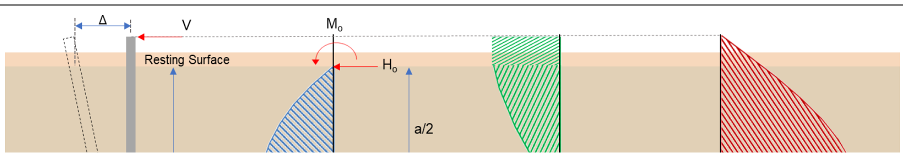
$Ratio$ - Lateral soil capacity

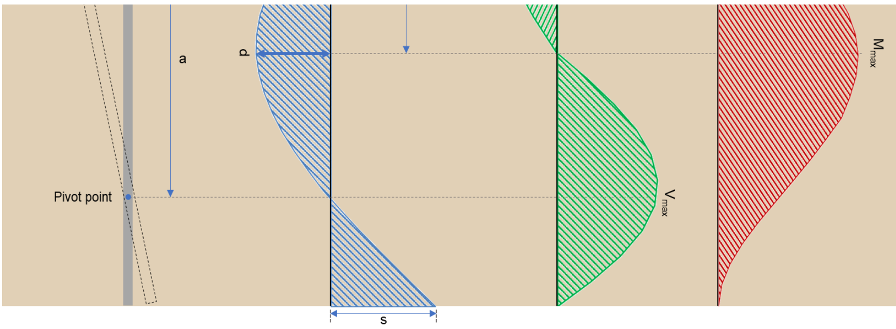
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{\left(1.0392\ \mathrm{kip/ft^2}\right)}{\left(1.05\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.9897$$

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





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

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.129\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.65748\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(59.126\text{ kipft}) + ((-4.129\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 9.415\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.415\text{ kipft/ft})}{(-0.65748\text{ kip/ft})}$$

$$E = 14.32\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.415\text{ kipft/ft}) \times (7\text{ ft})) + \left(3 \times (-0.65748\text{ kip/ft}) \times (7\text{ ft})^2\right)}{(6 \times (9.415\text{ kipft/ft})) + (4 \times (-0.65748\text{ kip/ft}) \times (7\text{ ft}))}$$

$$a = 4.81\text{ ft}$$

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

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

$$V_{max} = ((-0.65748\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (14.32\text{ ft})}{(7\text{ ft})} + 3\right) \times \left(\frac{(4.81\text{ ft})}{(7\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (14.32\text{ ft})}{(7\text{ ft})} + 2\right) \times \left(\frac{(4.81\text{ ft})}{(7\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 11.257\text{ kip}$$

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

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

$$M_{max} = ((-0.65748\text{ kip/ft}) \times (48\text{ in}) \times (7\text{ ft})) \times \left[\left(\frac{(14.32\text{ ft})}{(7\text{ ft})} + \frac{(4.81\text{ ft})}{2 \times (7\text{ ft})}\right) - \left[\left(\frac{4 \times (14.32\text{ ft})}{(7\text{ ft})} + 3\right) \times \left(\frac{(4.81\text{ ft})}{2 \times (7\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (14.32\text{ ft})}{(7\text{ ft})} + 2\right) \times \left(\frac{(4.81\text{ ft})}{2 \times (7\text{ ft})}\right)^4\right]\right]$$

	$$M_{max} = 37.723 \text{ kipft}$$	

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(6.585 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.377 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.377 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

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

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.585\ \mathrm{kip})}{(2675.2\ \mathrm{kip})}$$

$$Ratio = 0.0024615$$

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \mathrm{ksi} \rightarrow 2500\ \mathrm{psi}$, $P = 6.585\ \mathrm{kip} \rightarrow 6585\ \mathrm{lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \mathrm{psi})} + \frac{(6585\ \mathrm{lbf})}{6 \times \left(2304\ \mathrm{in}^2\right)}\right] \times (48\ \mathrm{in}) \times (38.4\ \mathrm{in})$$

$$V_{c,a} = 119.36\ \mathrm{kip}$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \mathrm{kip}), (119.36\ \mathrm{kip}), (348.89\ \mathrm{kip})\right]$$

$$V_c = 119.36\ \mathrm{kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.36\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 110.67\ \text{kip}$$

Considering x-direction:

$V_{max} = 11.257\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(11.257\ \text{kip})}{(110.67\ \text{kip})}$$

$$Ratio = 0.10172$$

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

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:

$M_{max} = 37.723\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(37.723\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.15113$$

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