Project Details

Project Name: MTSOLAR_20JC388A6H2AE-4x7=28 no override

Location: 253 Flatiron Trl, Cameron, MT 59720, USA

Unique ID: 3P-17-10TOP-SD-24-L-4Hx7W-FHL1

Dealer: ______________________________

Date: Tue Nov 26 2024

Number of Modules: 28

Number of Poles: 3

Date Sold:

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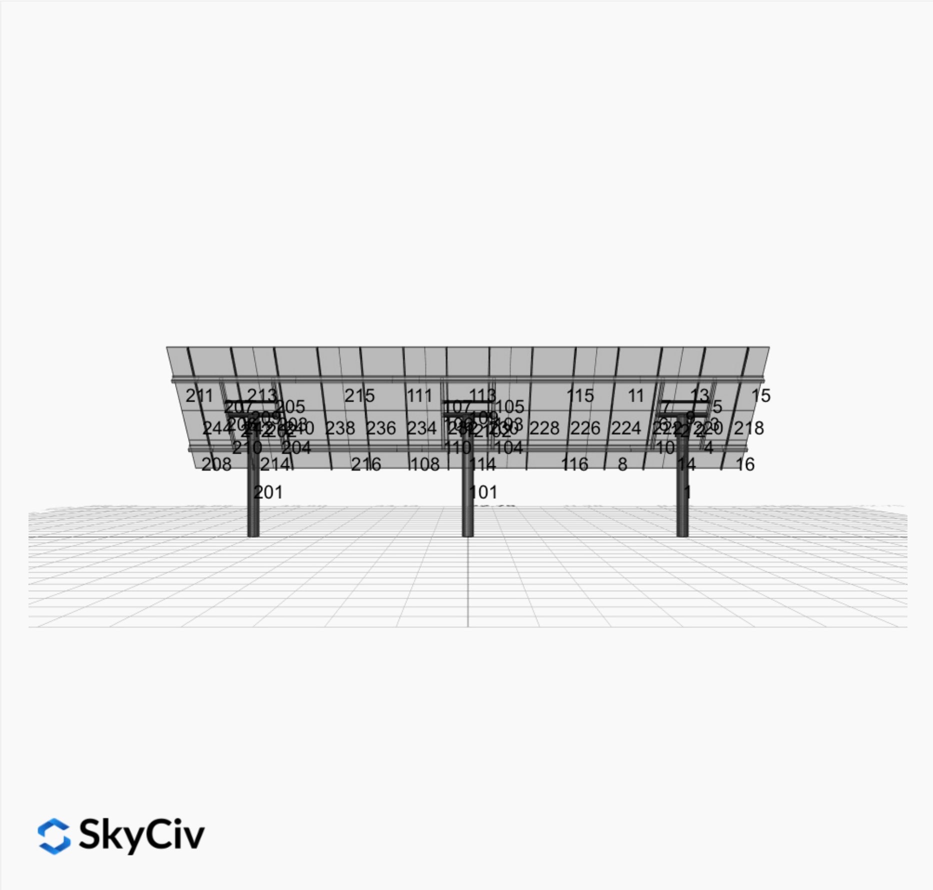


Array Dimensions N/S	13.17 ft
Array Dimensions E/W	45.50 ft
Winter Tilt Angle	45
Front Edge Clearance	5 ft

MT Solar Bill of Materials (3P-17-10TOP-SD-24-L-4Hx7W-FHL1)

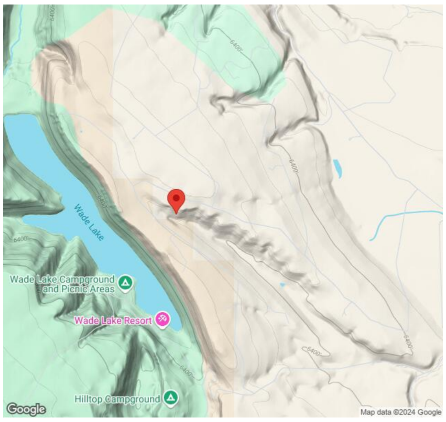
Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	3
MTS-HF-SD	H-Frame Assembly-SD	3
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	7

Rail Bill of Materials

Part	Qty
Rails (156in)	14
Rail Attachment	28
Module Mid Clamp	42

Part	Qty
Module End Clamp	28
Ground Lug	7

Site Details:



Site Address: 253 Flatiron Trl, Cameron, MT 59720, USA

Array Specification

Duty Classification:	SD
Module Width:	39.00 in
Module Length:	77.00in
Number of Rows:	4
Number of Columns:	7
Total Number of Modules:	28
Winter Tilt Angle:	45
Front Edge Clearance:	5
Total Array Height at Tilt:	14.31 ft
Total Frame Length:	45.50 ft
Frame Weight:	2967 lbs
Array Dimensions N/S:	13.17 ft
Array Dimensions E/W:	45.50 ft
Rail Length:	158.00 in
Rail Spacing:	3.25 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	9.66 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: 6.25 ft Pile 2: 6.75 ft Pile 3: 6.25 ft
Foundation Volume:	11.407 y^3

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	253 Flatiron Trl, Cameron, MT 59720, USA
Wind Speed:	110 mph
Snow Load:	30 psf

Design Disclaimer

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

AutoDesigner Input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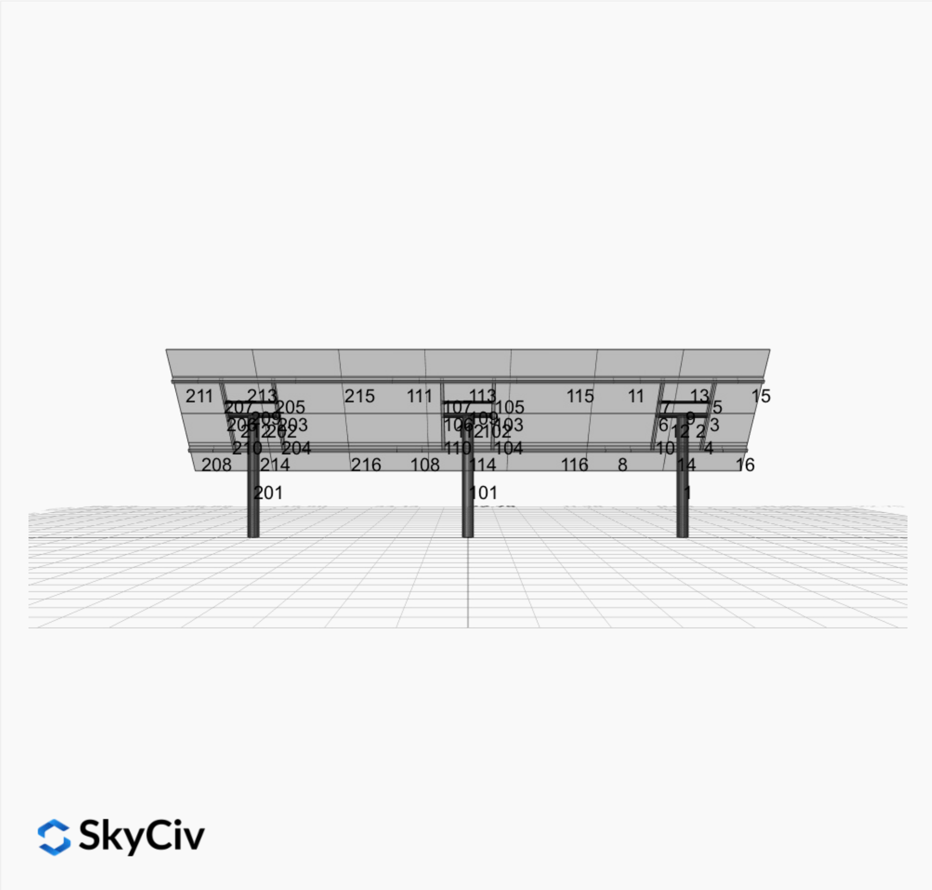
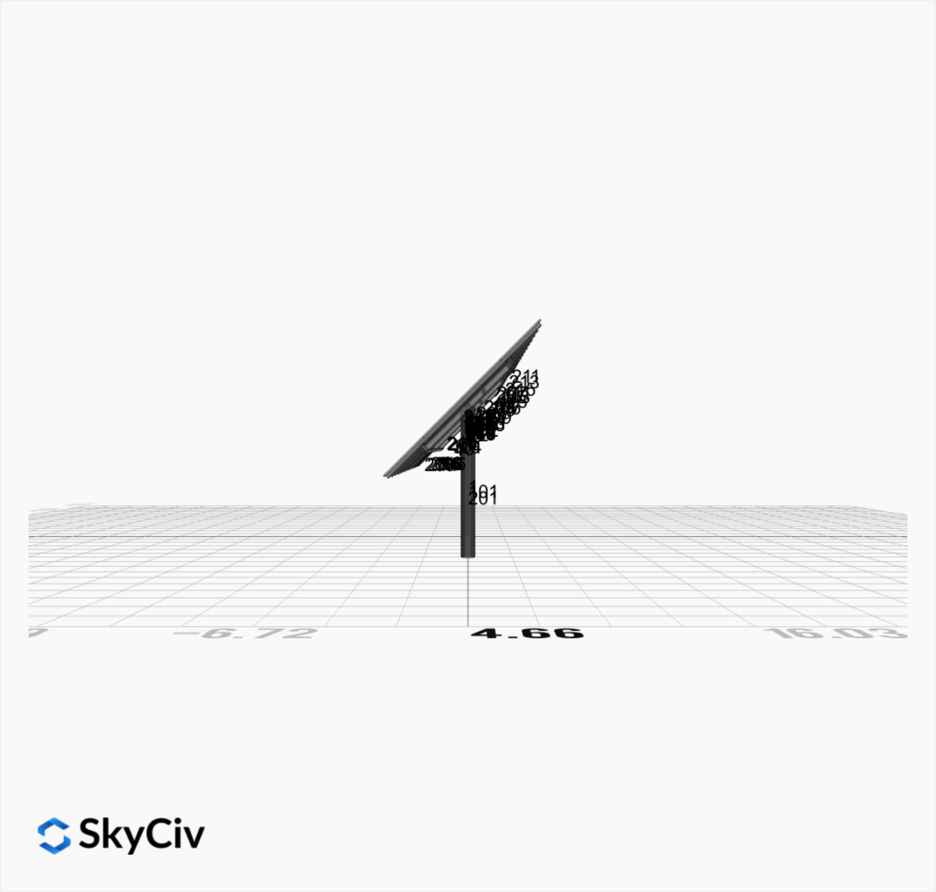
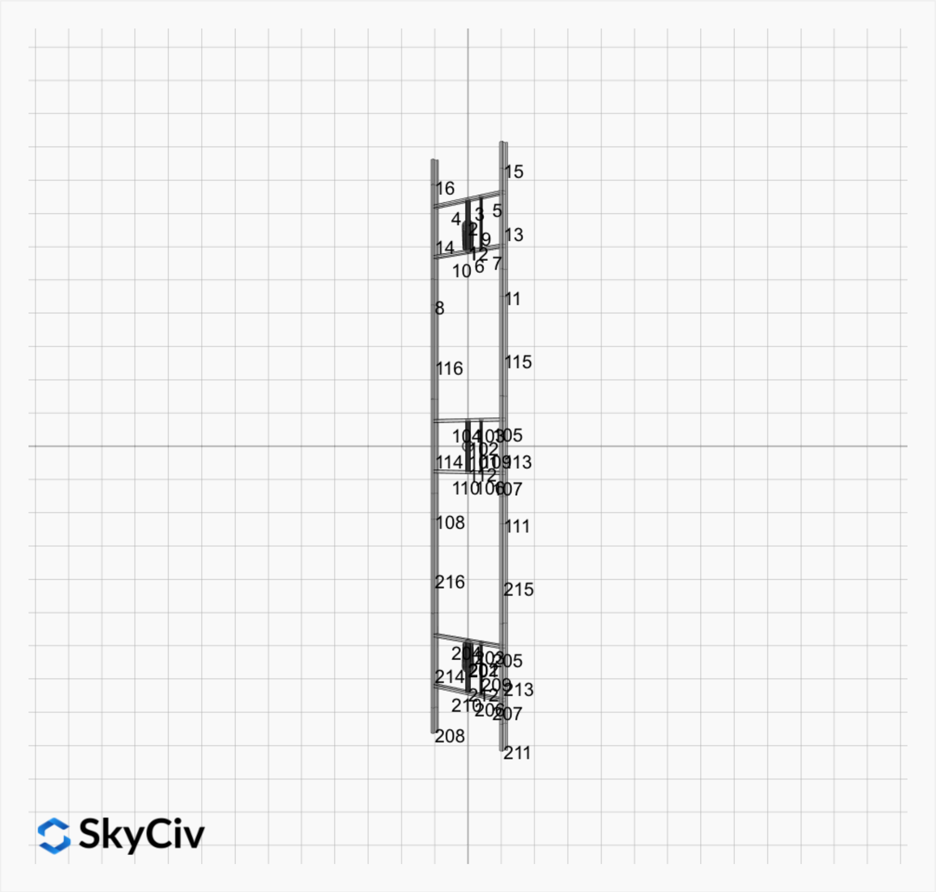
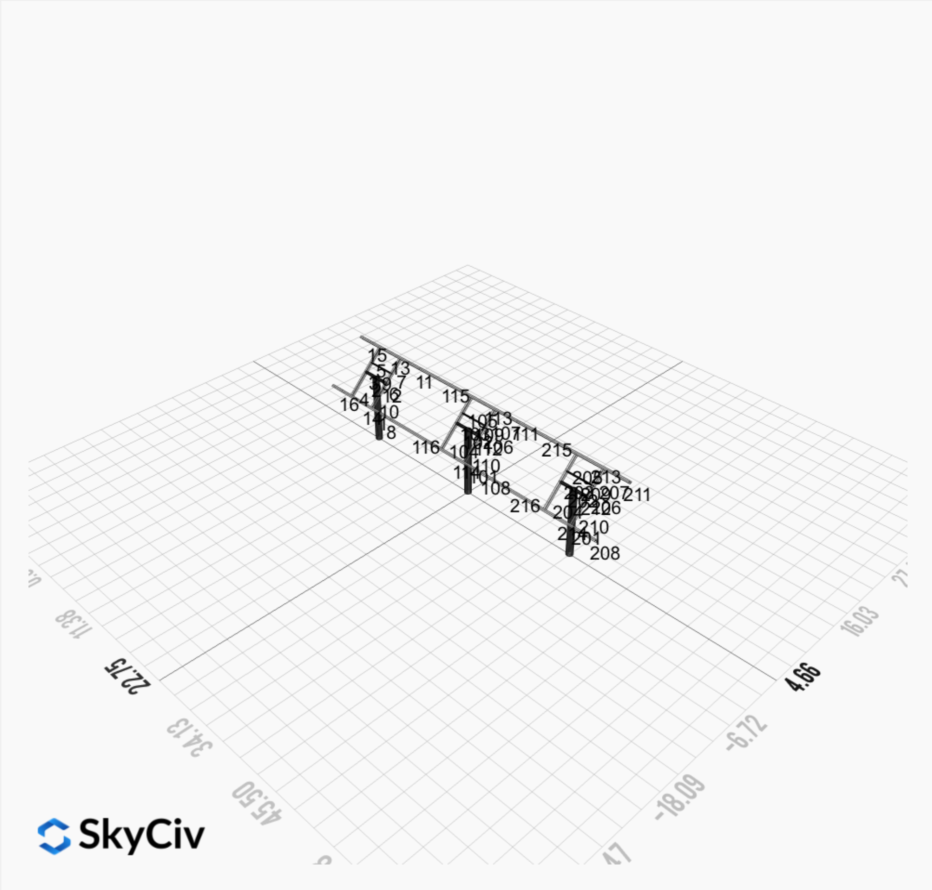
Design Notes:

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

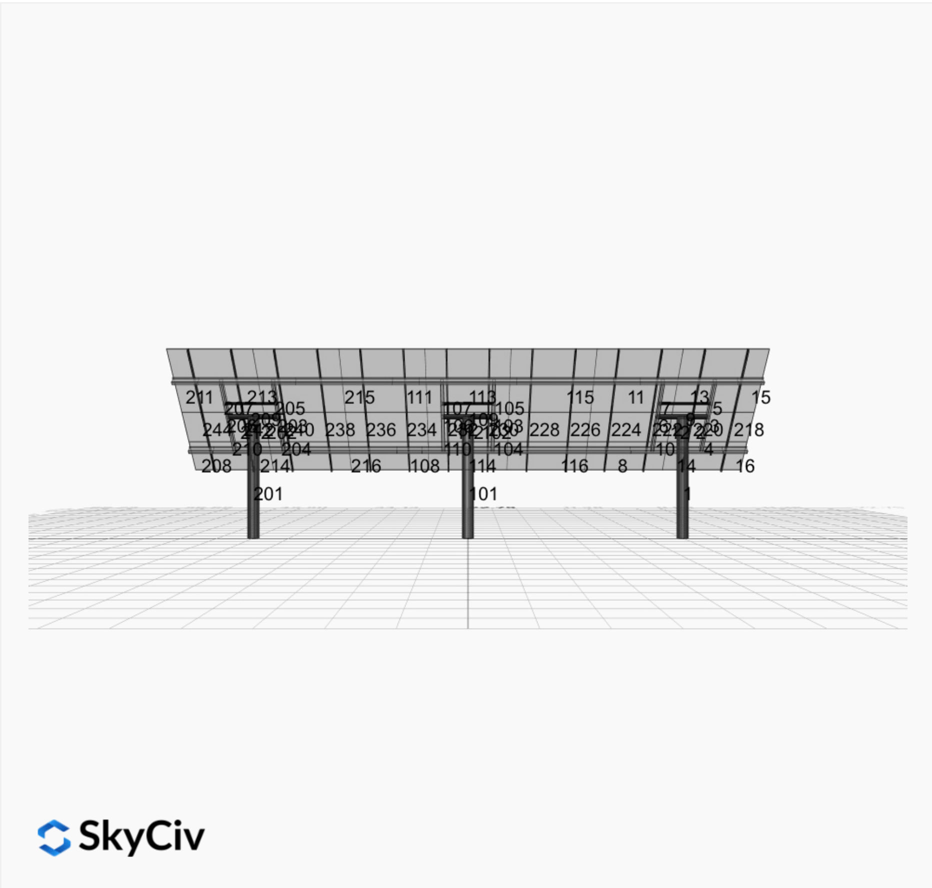






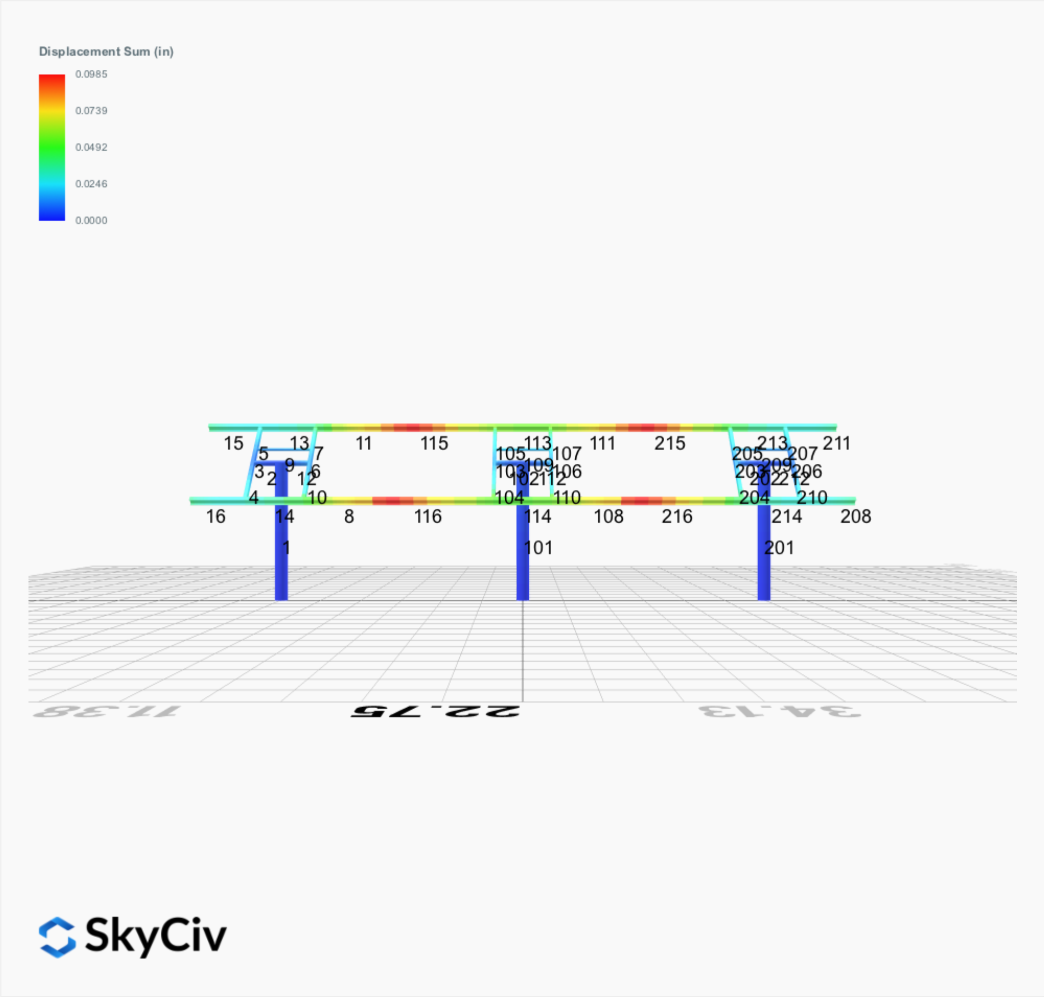


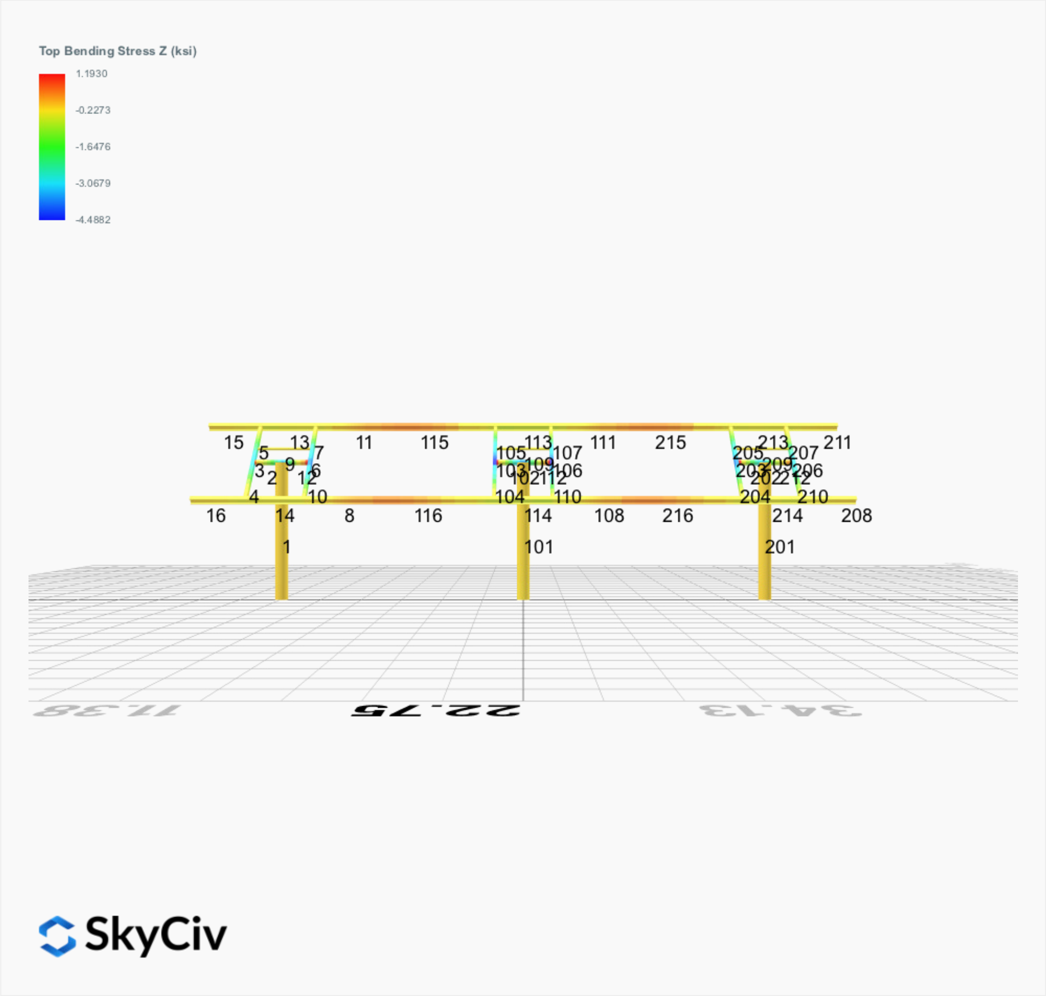
SkyCiv
11.38
22.75
34.13
45.50
16.03
4.66
-6.72
-18.09



SkyCiv

FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
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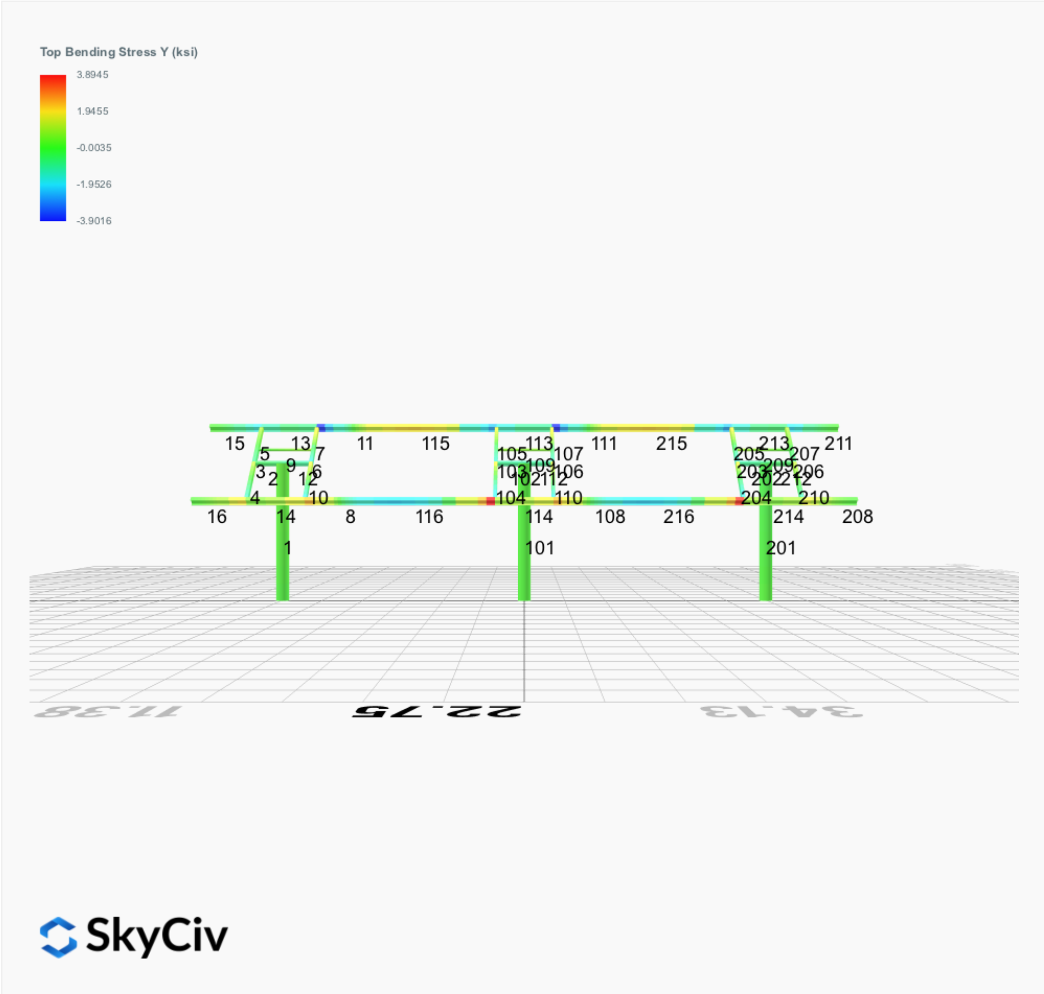
-0.2273
-1.6476
-3.0679
-4.4882
SkyCiv


Top Bending Stress Y (ksi)
3.8945
1.9455
-0.0035
-1.9526
-3.9016
15
13
7
11
115
113
107
111
215
213
207
211
5
9
105
109
205
209
3
6
103
106
203
206
2
12
102
112
202
212
4
10
104
110
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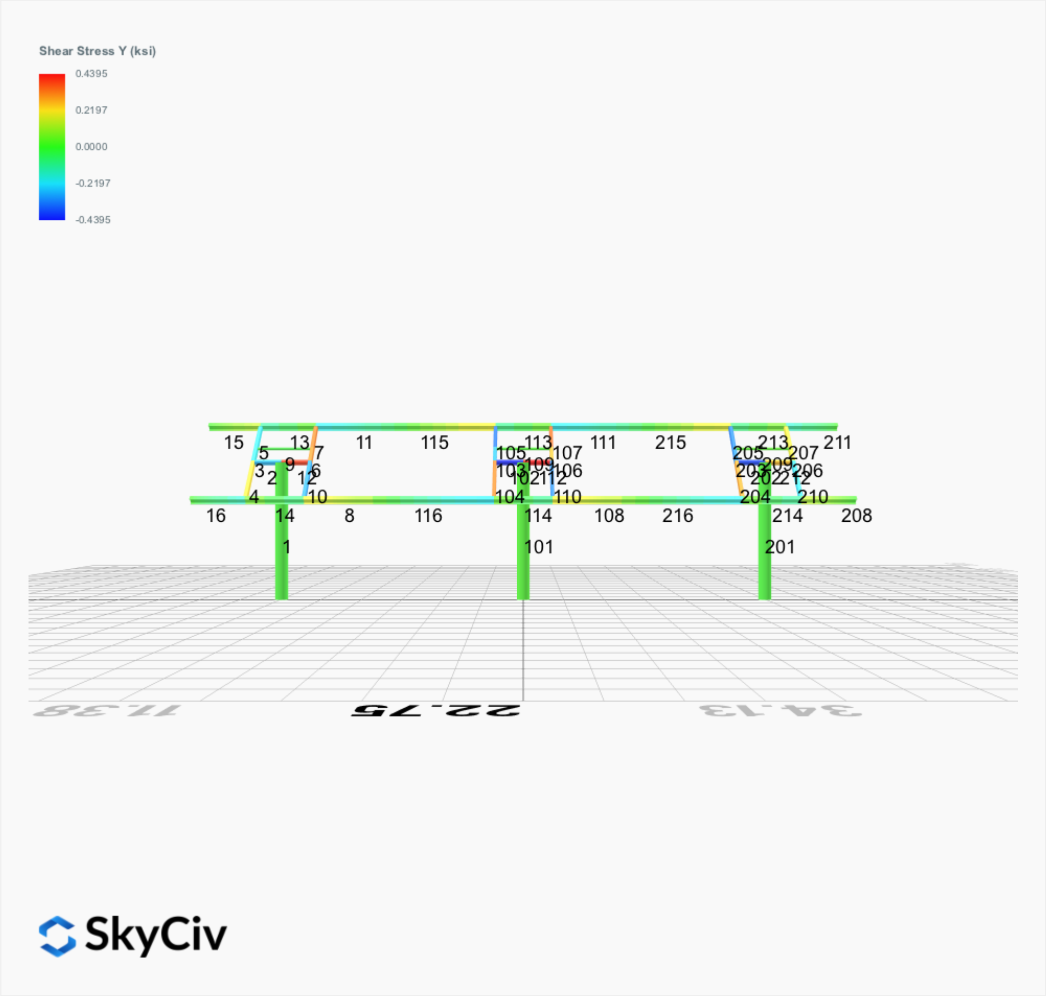
101
201
22.75
SkyCiv


Shear Stress Y (ksi)
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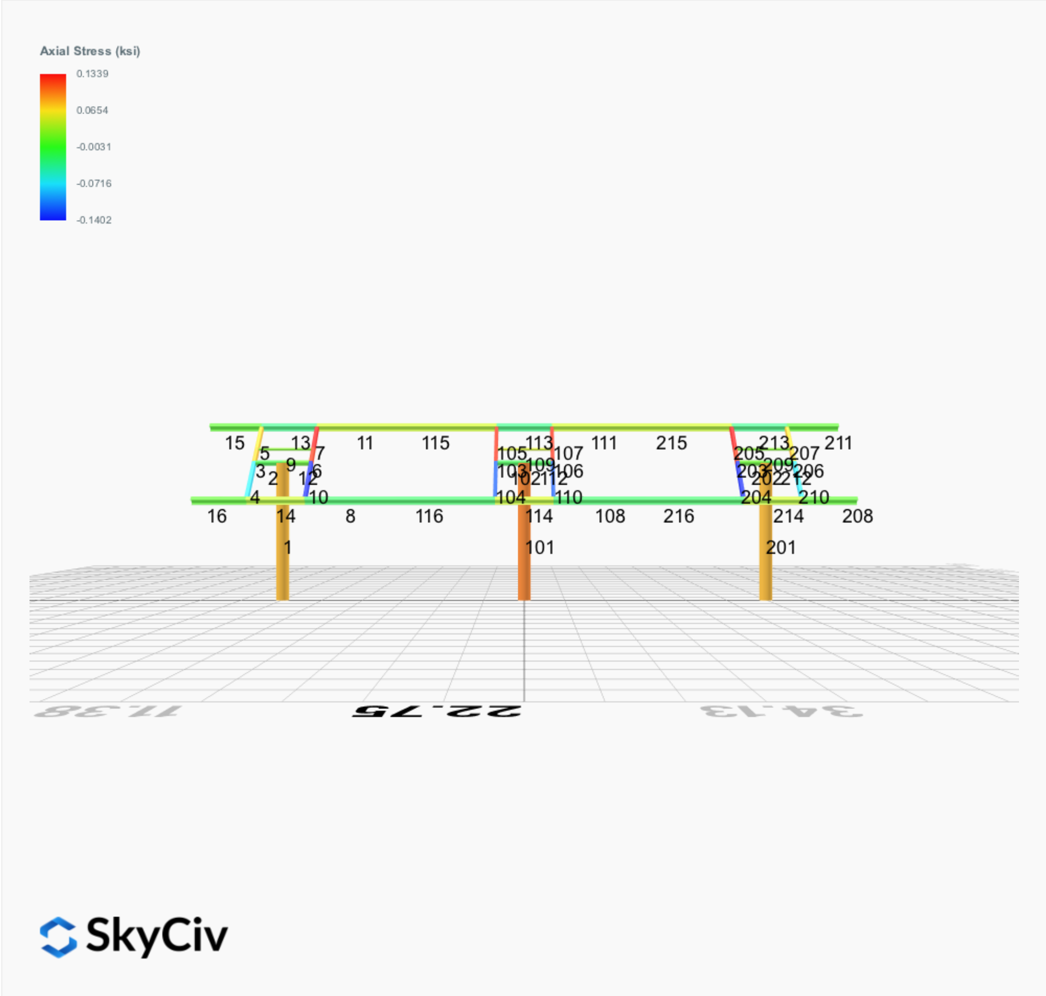
-0.2197
-0.4395
SkyCiv


Axial Stress (ksi)
0.1339
0.0654
-0.0031
-0.0716
-0.1402
22.75
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	0.0131	1.6669	0.0406	0.0671	-0.0340	-0.1053
ULS: 2. D + L	0.0131	1.6669	0.0406	0.0671	-0.0340	-0.1053
ULS: 3. D + (S or Lr or R)	0.0257	2.7472	0.0795	0.1318	-0.0671	-0.2201
ULS: 3. D + (S or Lr or R)	0.0131	1.6669	0.0406	0.0671	-0.0340	-0.1053
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0226	2.4772	0.0698	0.1156	-0.0588	-0.1914
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	1.6669	0.0406	0.0671	-0.0340	-0.1053
ULS: 5b. D + 0.7E	0.0131	1.6669	0.0406	0.0671	-0.0340	-0.1053
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0226	2.4772	0.0698	0.1156	-0.0588	-0.1914
ULS: 8. 0.6D + 0.7E	0.0079	1.0002	0.0243	0.0403	-0.0204	-0.0632
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7506	4.4083	0.1639	0.2012	-0.8748	27.7306
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.7506	4.4083	0.1639	0.2012	-0.8748	27.7306
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0098	-0.3151	-0.0469	-0.0284	0.5614	-18.7558
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7842	-0.0856	-0.0415	-0.0215	0.5329	-22.2230
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0502	4.5332	0.1623	0.2162	-0.6894	20.6856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0502	4.5332	0.1623	0.2162	-0.6894	20.6856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5201	0.9906	0.0042	0.0440	0.3877	-14.1793
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3509	1.1628	0.0083	0.0492	0.3664	-16.7797
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0596	3.7230	0.1330	0.1677	-0.6646	20.7716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0596	3.7230	0.1330	0.1677	-0.6646	20.7716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5107	0.1804	-0.0250	-0.0045	0.4125	-14.0932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3414	0.3526	-0.0210	0.0006	0.3912	-16.6936
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7558	3.7415	0.1477	0.1744	-0.8612	27.7728
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.7558	3.7415	0.1477	0.1744	-0.8612	27.7728
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0046	-0.9819	-0.0631	-0.0553	0.5750	-18.7137
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7789	-0.7523	-0.0577	-0.0484	0.5465	-22.1809

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	7.1094
Shear X	-4.6061
Shear Z	0.2741
Moment X	0.3369
Moment Y (Twist)	1.4600
Moment Z	46.3933

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	4.5332
Shear X	-2.7558
Shear Z	0.1639
Moment X	0.2162
Moment Y (Twist)	0.8748
Moment Z	27.7728

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	1.9305	0.0000	0.0000	0.0000	0.2602
ULS: 2. D + L	-0.0262	1.9305	0.0000	0.0000	0.0000	0.2602
ULS: 3. D + (S or Lr or R)	-0.0515	3.2636	0.0000	0.0000	0.0000	0.4974
ULS: 3. D + (S or Lr or R)	-0.0262	1.9305	0.0000	0.0000	0.0000	0.2602
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0452	2.9304	0.0000	0.0000	0.0000	0.4381

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0262	1.9305	0.0000	0.0000	0.0000	0.2602
ULS: 5b. D + 0.7E	-0.0262	1.9305	0.0000	0.0000	0.0000	0.2602
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0452	2.9304	0.0000	0.0000	0.0000	0.4381
ULS: 8. 0.6D + 0.7E	-0.0157	1.1583	0.0000	0.0000	0.0000	0.1561
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.4586	5.4076	0.0000	0.0000	0.0000	34.5715
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.4586	5.4076	0.0000	0.0000	0.0000	34.5715
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.4618	-0.5868	0.0000	0.0000	0.0000	-22.7892
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.1506	-0.2834	0.0000	0.0000	0.0000	-26.6539
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6195	5.5381	0.0000	0.0000	0.0000	26.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.6195	5.5381	0.0000	0.0000	0.0000	26.1716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8209	1.0423	0.0000	0.0000	0.0000	-16.8490
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.5875	1.2699	0.0000	0.0000	0.0000	-19.7475
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.6005	4.5383	0.0000	0.0000	0.0000	25.9937
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.6005	4.5383	0.0000	0.0000	0.0000	25.9937
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8398	0.0425	0.0000	0.0000	0.0000	-17.0269
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6064	0.2701	0.0000	0.0000	0.0000	-19.9254
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.4481	4.6354	0.0000	0.0000	0.0000	34.4674
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.4481	4.6354	0.0000	0.0000	0.0000	34.4674
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.4723	-1.3590	0.0000	0.0000	0.0000	-22.8933
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.1611	-1.0556	0.0000	0.0000	0.0000	-26.7580

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	8.7782
Shear X	-5.7642
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0002
Moment Z	57.8349

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	5.5381
Shear X	-3.4586
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	34.5715

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	1.6669	-0.0406	-0.0671	0.0340	-0.1053
ULS: 2. D + L	0.0131	1.6669	-0.0406	-0.0671	0.0340	-0.1053
ULS: 3. D + (S or Lr or R)	0.0257	2.7472	-0.0795	-0.1318	0.0671	-0.2201
ULS: 3. D + (S or Lr or R)	0.0131	1.6669	-0.0406	-0.0671	0.0340	-0.1053
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0226	2.4772	-0.0698	-0.1156	0.0588	-0.1914
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0131	1.6669	-0.0406	-0.0671	0.0340	-0.1053
ULS: 5b. D + 0.7E	0.0131	1.6669	-0.0406	-0.0671	0.0340	-0.1053
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0226	2.4772	-0.0698	-0.1156	0.0588	-0.1914
ULS: 8. 0.6D + 0.7E	0.0079	1.0002	-0.0243	-0.0403	0.0204	-0.0632
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.7506	4.4083	-0.1639	-0.2012	0.8748	27.7306
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.7506	4.4083	-0.1639	-0.2012	0.8748	27.7306
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.0098	-0.3151	0.0469	0.0284	-0.5614	-18.7558
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.7842	-0.0856	0.0415	0.0215	-0.5329	-22.2230

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0502	4.5332	-0.1623	-0.2162	0.6894	20.6856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0502	4.5332	-0.1623	-0.2162	0.6894	20.6856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5201	0.9906	-0.0042	-0.0440	-0.3877	-14.1793
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3509	1.1628	-0.0083	-0.0492	-0.3663	-16.7797
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0596	3.7230	-0.1330	-0.1677	0.6646	20.7716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.0596	3.7230	-0.1330	-0.1677	0.6646	20.7716
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.5107	0.1804	0.0250	0.0045	-0.4125	-14.0932
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.3414	0.3526	0.0210	-0.0006	-0.3912	-16.6936
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.7558	3.7415	-0.1477	-0.1744	0.8612	27.7728
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.7558	3.7415	-0.1477	-0.1744	0.8612	27.7728
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.0046	-0.9819	0.0631	0.0553	-0.5750	-18.7137
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.7789	-0.7523	0.0577	0.0484	-0.5465	-22.1809

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	7.1095
Shear X	-4.6061
Shear Z	-0.2741
Moment X	-0.3367
Moment Y (Twist)	1.4603
Moment Z	46.3933

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	4.5332
Shear X	-2.7558
Shear Z	-0.1639
Moment X	-0.2162
Moment Y (Twist)	0.8748
Moment Z	27.7728

Project Details

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

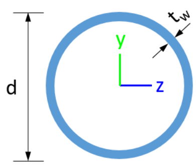
User Name: sales@mtsolar.us
Unit System: imperial

Design Input Information

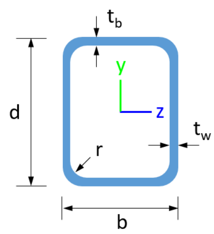
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

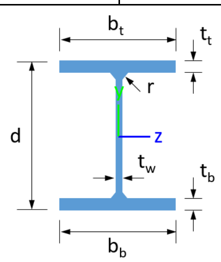
Section Dimensions



ID	Name	d (in)	t_w (in)					
1	2in Pipe Sch 40	2.38	0.15					
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	20.28	20.28	9.66	-	300	200	1
2	4	1.30	1.30	2.00	-	300	200	1
3	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,2.23,1.16,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17	300	200	1
4	15	2.44	2.44	3.75	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.67,1.66,1.62,1.67,1.67,1.66,1.64,1.67,1.67,1.60,1.68,1.67,1.67,1.65,1.86,1.67,1.67,1.66,1.65	300	200	1
5	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.12,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	300	200	1
6	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.04,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
7	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.53,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
8	18	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.33,1.33,1.33,1.22,1.33,1.33,1.33,1.26,1.33,1.33,1.33,1.37,1.33,1.33,1.33,1.18,1.33,1.33,1.33,1.27	300	200	1
9	1	2.60	2.60	4.00	-	300	200	1
10	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.65,1.67,1.67,1.61,1.68,1.67,1.67,1.66,1.33,1.67,1.67,1.66,1.66	300	200	1
11	18	1.33	1.33	2.05	1.35,1.35,1.35,1.35,1.35,1.35,1.36,1.36,1.37,1.38,1.36,1.36,1.36,1.38,1.36,1.36,1.87,1.44,1.36,1.36,1.37,1.39,1.36,1.36,1.36,1.38	300	200	1
12	4	4.20	1.30	2.00	-	300	200	1
13	18	4.88	4.00	7.50	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.20,1.17,1.17,1.15,1.20,1.17,1.17,1.22,1.29,1.17,1.17,1.15,1.21,1.17,1.17,1.15,1.20	300	200	1
14	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.17,1.24,1.15,1.15,1.17,1.18,1.16,1.16,1.21,1.30,1.15,1.15,1.17,1.54,1.15,1.15,1.17,1.15	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	20.28	20.28	9.66	-	300	200	1
102	4	1.30	1.30	2.00	-	300	200	1
103	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.90,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
104	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.65,1.67,1.67,1.62,1.68,1.67,1.67,1.66,1.16,1.67,1.67,1.66,1.66	300	200	1
105	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.11,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
106	15	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.90,1.17,1.18,1.18,1.17,1.17,1.18,1.18,1.18,1.18	300	200	1
107	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.11,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
108	18	1.33	1.33	2.05	1.66,1.66,1.66,1.66,1.66,1.66,1.65,1.65,1.62,2.10,1.65,1.65,1.62,2.05,1.65,1.65,1.46,1.54,1.65,1.65,1.61,1.22,1.65,1.65,1.62,2.04	300	200	1
109	1	2.60	2.60	4.00	-	300	200	1
110	15	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.65,1.67,1.67,1.62,1.68,1.67,1.67,1.66,1.16,1.67,1.67,1.66,1.66	300	200	1
111	18	1.33	1.33	2.05	1.62,1.62,1.62,1.62,1.62,1.62,1.52,1.52,1.48,1.44,1.52,1.52,1.50,1.45,1.54,1.54,1.02,1.29,1.53,1.53,1.46,1.42,1.51,1.51,1.51,1.46	300	200	1
112	4	1.30	1.30	2.00	-	300	200	1

113	18	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.08,1.06,1.06,1.06,1.07,1.06,1.06,1.17,1.29,1.06,1.06,1.07,1.08,1.06,1.06,1.06,1.07	300	200	1
114	18	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.03,1.05,1.05,1.06,1.03,1.05,1.05,1.08,1.09,1.05,1.05,1.06,1.00,1.05,1.05,1.06,1.04	300	200	1
115	18	4.84	4.84	7.45	1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.13,1.12,1.11,1.13,1.13,1.12,1.11,1.13,1.13,1.20,1.08,1.13,1.13,1.12,1.11,1.12,1.12,1.12,1.12	300	200	1
116	18	4.84	4.84	7.45	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.26,1.14,1.14,1.14,1.21,1.14,1.14,1.12,1.13,1.14,1.14,1.14,2.29,1.14,1.14,1.14,1.20	300	200	1
201	11	20.28	20.28	9.66	-	300	200	1
202	4	4.20	1.30	2.00	-	300	200	1
203	15	0.92	0.92	1.42	1.19,1.19,1.19,1.19,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.04,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18	300	200	1
204	15	2.44	2.44	3.75	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.65,1.67,1.67,1.61,1.68,1.67,1.67,1.66,1.33,1.67,1.67,1.66,1.66	300	200	1
205	15	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67,1.67,1.67,1.53,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.67	300	200	1
206	15	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,2.24,1.16,1.18,1.18,1.16,1.17,1.17,1.17,1.17,1.17	300	200	1
207	15	1.52	1.52	2.33	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.12,1.65,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66	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	1.69,1.68,1.69,1.68,1.68,1.69,1.67,1.67,1.66,1.62,1.67,1.67,1.66,1.64,1.67,1.67,1.60,1.68,1.67,1.67,1.65,1.86,1.67,1.67,1.66,1.65	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	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.20,1.17,1.17,1.15,1.20,1.17,1.17,1.22,1.29,1.17,1.17,1.15,1.21,1.17,1.17,1.15,1.20	300	200	1
214	18	4.88	4.00	7.50	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.17,1.24,1.15,1.15,1.17,1.18,1.16,1.16,1.21,1.30,1.15,1.15,1.17,1.54,1.15,1.15,1.17,1.15	300	200	1
215	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.09,1.47,1.12,1.09,1.09,1.10,1.10,1.10,1.10,1.10,1.10	300	200	1
216	18	4.84	4.84	7.45	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,2.03,1.09,1.09,1.09,1.07	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	535.87	388.86	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	6.26	29.14	16.61
4	79.65	72.01	10.99	6.26	29.14	16.61
5	79.65	73.44	10.99	6.26	29.14	16.61
6	79.65	74.02	10.99	6.26	29.14	16.61
7	79.65	73.44	10.99	6.26	29.14	16.61
8	120.60	115.40	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	6.26	29.14	16.61
11	120.60	115.40	23.36	6.45	30.09	45.74
12	142.83	131.65	16.17	16.17	42.85	42.85
13	120.60	84.03	20.19	6.45	30.09	45.74
14	120.60	84.03	20.28	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	388.86	147.68	147.68	160.76	160.76

101	535.87	388.86	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	6.26	29.14	16.61
104	79.65	72.01	10.99	6.26	29.14	16.61
105	79.65	73.44	10.99	6.26	29.14	16.61
106	79.65	74.02	10.99	6.26	29.14	16.61
107	79.65	73.44	10.99	6.26	29.14	16.61
108	120.60	115.40	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	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.58	6.45	30.09	45.74
114	120.60	84.03	17.66	6.45	30.09	45.74
115	120.60	84.26	19.10	6.45	30.09	45.74
116	120.60	84.26	19.76	6.45	30.09	45.74
201	535.87	388.86	147.68	147.68	160.76	160.76
202	142.83	131.65	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	6.26	29.14	16.61
204	79.65	72.01	10.99	6.26	29.14	16.61
205	79.65	73.44	10.99	6.26	29.14	16.61
206	79.65	74.02	10.99	6.26	29.14	16.61
207	79.65	73.44	10.99	6.26	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	6.26	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	84.03	20.19	6.45	30.09	45.74
214	120.60	84.03	20.28	6.45	30.09	45.74
215	120.60	84.26	19.26	6.45	30.09	45.74
216	120.60	84.26	19.00	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	0.018	0.314	0.016	0.029	0.002	0.325	#13	0.331	Not Required	Pass
2	0.001	0.276	0.223	0.066	0.047	0.499	#13	0.052	Not Required	Pass
3	0.007	0.623	0.046	0.061	0.009	0.645	#13	0.044	Not Required	Pass
4	0.005	0.553	0.071	0.056	0.014	0.614	#13	0.078	Not Required	Pass
5	0.006	0.387	0.055	0.062	0.010	0.393	#13	0.073	Not Required	Pass
6	0.009	0.801	0.099	0.082	0.022	0.872	#13	0.044	Not Required	Pass
7	0.010	0.497	0.119	0.080	0.023	0.517	#13	0.073	Not Required	Pass
8	0.002	0.109	0.055	0.039	0.008	0.148	#13	0.088	Not Required	Pass
9	0.001	0.058	0.079	0.002	0.003	0.136	#13	0.198	Not Required	Pass
10	0.009	0.708	0.115	0.072	0.021	0.733	#13	0.078	Not Required	Pass
11	0.002	0.120	0.053	0.044	0.008	0.156	#13	0.088	Not Required	Pass
12	0.001	0.418	0.313	0.089	0.060	0.732	#13	0.167	Not Required	Pass
13	0.003	0.128	0.164	0.062	0.011	0.224	#21	0.265	Not Required	Pass
14	0.003	0.116	0.163	0.054	0.011	0.209	#21	0.177	Not Required	Pass
15	0.000	0.025	0.024	0.020	0.003	0.041	#21	Not Required	Not Required	Pass

16	0.000	0.023	0.024	0.018	0.003	0.040	#21	Not Required	Not Required	Pass
101	0.023	0.392	0.000	0.036	0.000	0.403	#13	0.331	Not Required	Pass
102	0.001	0.436	0.340	0.097	0.067	0.777	#13	0.052	Not Required	Pass
103	0.009	0.882	0.077	0.089	0.016	0.933	#13	0.044	Not Required	Pass
104	0.009	0.805	0.119	0.081	0.021	0.869	#13	0.078	Not Required	Pass
105	0.009	0.548	0.122	0.088	0.023	0.567	#13	0.073	Not Required	Pass
106	0.009	0.882	0.077	0.089	0.016	0.933	#13	0.044	Not Required	Pass
107	0.009	0.548	0.122	0.088	0.023	0.567	#13	0.073	Not Required	Pass
108	0.002	0.086	0.053	0.045	0.008	0.112	#13	0.088	Not Required	Pass
109	0.003	0.057	0.063	0.001	0.000	0.121	#13	0.198	Not Required	Pass
110	0.009	0.805	0.119	0.081	0.021	0.869	#13	0.078	Not Required	Pass
111	0.002	0.106	0.052	0.048	0.008	0.131	#13	0.088	Not Required	Pass
112	0.001	0.436	0.340	0.097	0.067	0.777	#13	0.052	Not Required	Pass
113	0.003	0.175	0.167	0.066	0.011	0.278	#21	0.265	Not Required	Pass
114	0.004	0.185	0.168	0.060	0.011	0.281	#21	0.265	Not Required	Pass
115	0.003	0.186	0.095	0.048	0.008	0.244	#13	0.321	Not Required	Pass
116	0.002	0.160	0.095	0.045	0.008	0.219	#13	0.321	Not Required	Pass
201	0.018	0.314	0.016	0.029	0.002	0.325	#13	0.331	Not Required	Pass
202	0.001	0.418	0.313	0.089	0.060	0.732	#13	0.167	Not Required	Pass
203	0.009	0.801	0.099	0.082	0.022	0.872	#13	0.044	Not Required	Pass
204	0.009	0.708	0.115	0.072	0.021	0.733	#13	0.078	Not Required	Pass
205	0.010	0.497	0.119	0.080	0.023	0.517	#13	0.073	Not Required	Pass
206	0.007	0.623	0.046	0.061	0.009	0.645	#13	0.044	Not Required	Pass
207	0.006	0.387	0.055	0.062	0.010	0.393	#13	0.073	Not Required	Pass
208	0.000	0.023	0.024	0.018	0.003	0.040	#21	Not Required	Not Required	Pass
209	0.001	0.058	0.079	0.002	0.003	0.136	#13	0.198	Not Required	Pass
210	0.005	0.553	0.071	0.056	0.014	0.614	#13	0.078	Not Required	Pass
211	0.000	0.025	0.024	0.020	0.003	0.041	#21	Not Required	Not Required	Pass
212	0.001	0.276	0.223	0.066	0.047	0.499	#13	0.052	Not Required	Pass
213	0.003	0.128	0.164	0.062	0.011	0.224	#21	0.177	Not Required	Pass
214	0.003	0.116	0.163	0.054	0.011	0.209	#21	0.265	Not Required	Pass
215	0.003	0.192	0.095	0.044	0.008	0.250	#13	0.321	Not Required	Pass
216	0.002	0.170	0.095	0.039	0.008	0.229	#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

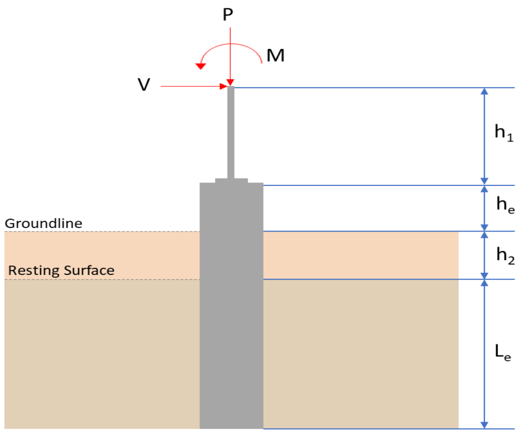
SkyCiv Foundation Design

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

$L = 6.25\ \mathrm{ft}$ - 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) $(\mathrm{psf/ft})$
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.533	7.109
V_x (kip)	-2.756	-4.606
V_z (kip)	0.164	0.274
M_x (kipft)	0.216	0.337
M_z (kipft)	27.773	46.393

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,

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

$$H_o = \frac{(-2.756\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.43885\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.773\ \text{kipft}) + ((-2.756\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.4225\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8466\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.164\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.026115\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.216\ \text{kipft}) + ((0.164\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.034395\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.7673\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8466\ \text{ft}), (1.7673\ \text{ft})]$$

$$L_{e,req} = 5.847\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.847\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.93552$$

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

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

$$q = \frac{(4.533 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28331 \text{ kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.28331 \text{ kip/ft}^2\right)}{(2000 \text{ psf})}$$

$$Ratio = 0.14166$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.43885 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 4.4225 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4225 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (-0.43885 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (4.4225 \text{ kipft/ft})) + (4 \times (-0.43885 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.319 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.4225 \text{ kipft/ft})) + (3 \times (-0.43885 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.4225 \text{ kipft/ft})) + (2 \times (-0.43885 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.22087 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.4225 \text{ kipft/ft})) + ((-0.43885 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.93728 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.319 \text{ ft})}{2}$$

$$p_a = 0.32393 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22087 \text{ kip/ft}^2\right)}{\left(0.32393 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.68184$$

Status: **PASS**
Ratio: **0.680**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.93728 \text{ kip/ft}^2\right)}{\left(0.9375 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.99976$$

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = 0.026115 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 0.034395 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034395 \text{ kipft/ft}) \times (6.25 \text{ ft})) + \left(3 \times (0.026115 \text{ kip/ft}) \times (6.25 \text{ ft})^2\right)}{(6 \times (0.034395 \text{ kipft/ft})) + (4 \times (0.026115 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5624 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.034395 \text{ kipft/ft})) + (3 \times (0.026115 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.034395 \text{ kipft/ft})) + (2 \times (0.026115 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.017582 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.034395 \text{ kipft/ft})) + ((0.026115 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.035636 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5624 \text{ ft})}{2}$$

$$p_a = 0.34218 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.017582 \text{ kip/ft}^2\right)}{\left(0.34218 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.051383$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

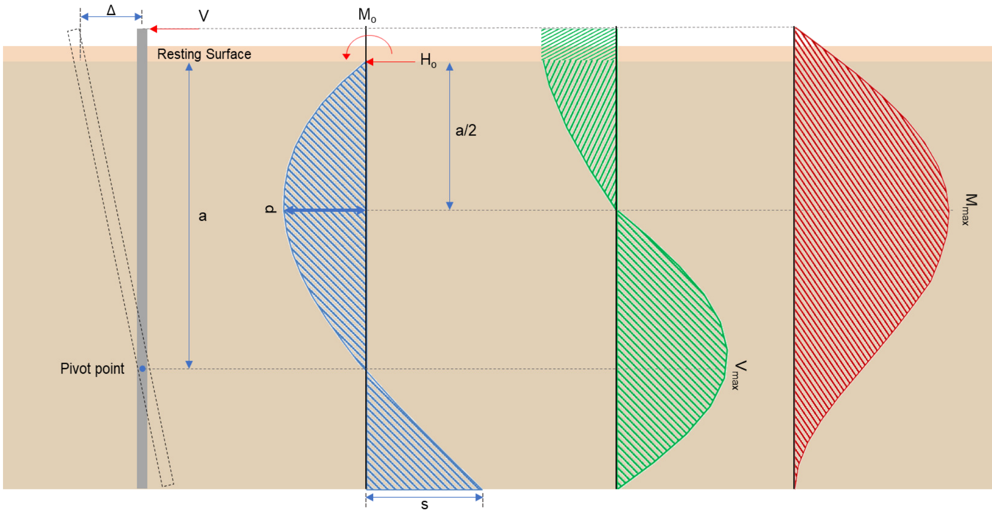
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.035636\ \mathrm{kip/ft^2})}{(0.9375\ \mathrm{kip/ft^2})}$$

$$Ratio = 0.038012$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.606\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.73344\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.393\ \mathrm{kipft}) + ((-4.606\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 7.3874\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.3874\ \mathrm{kipft/ft})}{(-0.73344\ \mathrm{kip/ft})}$$

$$E = 10.072\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.3874\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.73344\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{\ldots}$$

$$a = \frac{}{(6 \times (7.3874\ \text{kipft/ft})) + (4 \times (-0.73344\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3191\ \text{ft}$$

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

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

$$V_{max} = ((-0.73344\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3191\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3191\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.301\ \text{kip}$$

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

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

$$M_{max} = ((-0.73344\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(10.072\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 30.526\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.274\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = 0.043631\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.337\ \text{kipft}) + ((0.274\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.053662\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.053662\ \text{kipft/ft})}{(0.043631\ \text{kip/ft})}$$

$$E = 1.2299\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.053662\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (0.043631\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.053662\ \text{kipft/ft})) + (4 \times (0.043631\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.5688\ \text{ft}$$

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

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

$$V_{max} = ((0.043631\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.5688\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.5688\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.17867\ \text{kip}$$

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

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

$$M_{max} = ((0.043631\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(1.2299\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.46205\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.109\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.36\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.36\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.109\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0026574$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.109 \text{ kip} \rightarrow 7109 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7109 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.43 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.43 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.43 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.43 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.71 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.301 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.301 \text{ kip})}{(110.71 \text{ kip})}$$

$$Ratio = 0.09304$$

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

Considering z-direction:

$V_{max} = 0.17867 \text{ kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.17867 \text{ kip})}{(110.71 \text{ kip})}$$

$$Ratio = 0.0016138$$

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:

$M_{max} = 30.526 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(30.526 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.1223$$

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

Considering z-direction:

$M_{max} = 0.46205 \text{ kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

		$$Ratio = \frac{(0.46205\ \text{kipft})}{(249.6\ \text{kipft})}$$ $$Ratio = 0.0018512$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

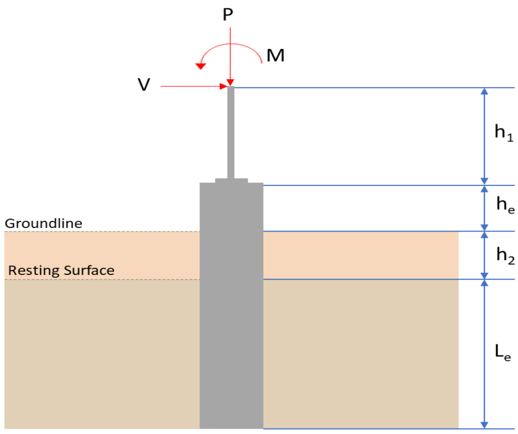
SkyCiv Foundation Design

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

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.533	7.109
V_x (kip)	-2.756	-4.606
V_z (kip)	-0.164	-0.274
M_x (kipft)	-0.216	-0.337
M_z (kipft)	27.773	46.393

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.756 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.773\ \text{kipft}) + ((-2.756\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 4.4225\ \text{kipft/ft}$$

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:
$L_{e,x} = 5.8466\ \text{ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.164\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.026115\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.216\ \text{kipft}) + ((-0.164\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.034395\ \text{kipft/ft}$$

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:
$L_{e,z} = 1.0394\ \text{ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8466\ \text{ft}), (1.0394\ \text{ft})]$$

$$L_{e,req} = 5.847\ \text{ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 6.25\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.847\ \text{ft})}{(6.25\ \text{ft})}$$

$$Ratio = 0.93552$$

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

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

$$q = \frac{(4.533\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.28331\ \mathrm{kip/ft^2}$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{\left(0.28331\ \mathrm{kip/ft^2}\right)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.14166$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25\ \mathrm{ft})}{(48\ \mathrm{in})}$$

$$L/D = 1.5625$$

Since L/D $\leq$ 10,

Pile is short.

Considering x-direction:

$H_o = -0.43885\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 4.4225\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4225\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.43885\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (4.4225\ \mathrm{kipft/ft})) + (4 \times (-0.43885\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.319\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.4225\ \mathrm{kipft/ft})) + (3 \times (-0.43885\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (4.4225\ \mathrm{kipft/ft})) + (2 \times (-0.43885\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = 0.22087\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (4.4225\ \mathrm{kipft/ft})) + ((-0.43885\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = 0.93728\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.319\ \mathrm{ft})}{2}$$

$$p_a = 0.32393\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.22087\ \mathrm{kip/ft^2}\right)}{\left(0.32393\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.68184$$

Status: **PASS**
Ratio: **0.680**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.93728\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = 0.99976$$

Status: **PASS**
Ratio: **1.000**

Considering z-direction:

$H_o = -0.026115\ \mathrm{kip/ft}$ - Lateral force per length of pile,
$M_o = 0.034395\ \mathrm{kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034395\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.026115\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{(6 \times (0.034395\ \mathrm{kipft/ft})) + (4 \times (-0.026115\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))}$$

$$a = 4.5624\ \mathrm{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.034395\ \mathrm{kipft/ft})) + (3 \times (-0.026115\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]^{\ 2}}{(6.25\ \mathrm{ft})^2 \times [(3 \times (0.034395\ \mathrm{kipft/ft})) + (2 \times (-0.026115\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}$$

$$p = -0.01066\ \mathrm{kip/ft^2}$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (0.034395\ \mathrm{kipft/ft})) + ((-0.026115\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft}))]}{(6.25\ \mathrm{ft})^2}$$

$$s = -0.014504\ \mathrm{kip/ft^2}$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150\ \mathrm{psf/ft}) \times \frac{(4.5624\ \mathrm{ft})}{2}$$

$$p_a = 0.34218\ \mathrm{kip/ft^2}$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.01066\ \mathrm{kip/ft^2}\right)}{\left(0.34218\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.031154$$

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

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\,L_e$$

$$p_s = (150\ \mathrm{psf/ft}) \times (6.25\ \mathrm{ft})$$

$$p_s = 0.9375\ \mathrm{kip/ft^2}$$

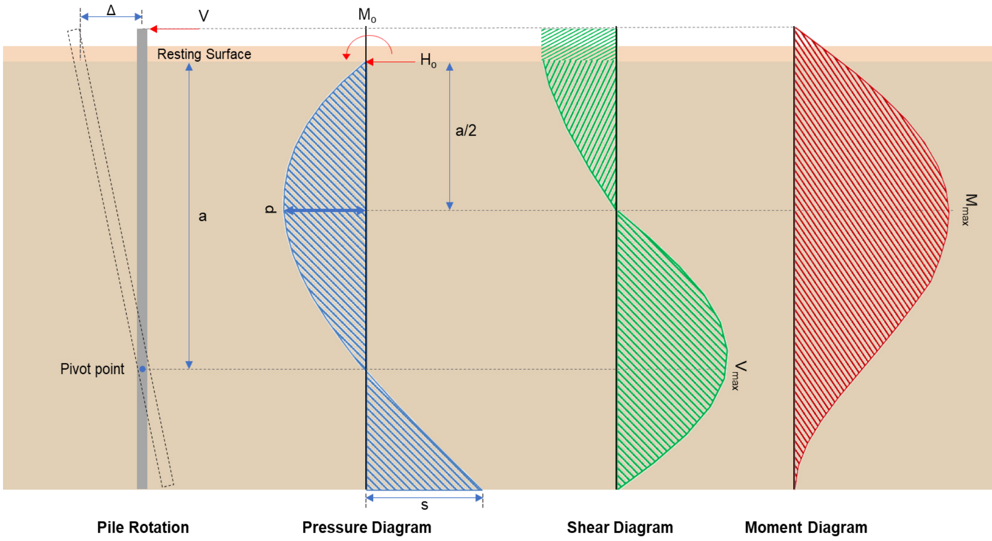
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(-0.014504\ \mathrm{kip/ft^2}\right)}{\left(0.9375\ \mathrm{kip/ft^2}\right)}$$

$$Ratio = -0.015471$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.606\ \mathrm{kip})}{1.57 \times (48\ \mathrm{in})}$$

$$H_o = -0.73344\ \mathrm{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.393\ \mathrm{kipft}) + ((-4.606\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 7.3874\ \mathrm{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.3874\ \mathrm{kipft/ft})}{(-0.73344\ \mathrm{kip/ft})}$$

$$E = 10.072\ \mathrm{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.3874\ \mathrm{kipft/ft}) \times (6.25\ \mathrm{ft})) + \left(3 \times (-0.73344\ \mathrm{kip/ft}) \times (6.25\ \mathrm{ft})^2\right)}{}$$

$$a = \frac{}{(6 \times (7.3874\ \text{kipft/ft})) + (4 \times (-0.73344\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.3191\ \text{ft}$$

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

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

$$V_{max} = ((-0.73344\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3191\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3191\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 10.301\ \text{kip}$$

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

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

$$M_{max} = ((-0.73344\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(10.072\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.072\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.3191\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 30.526\ \text{kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.274\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.043631\ \text{kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.337\ \text{kipft}) + ((-0.274\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 0.053662\ \text{kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.053662\ \text{kipft/ft})}{(-0.043631\ \text{kip/ft})}$$

$$E = 1.2299\ \text{ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.053662\ \text{kipft/ft}) \times (6.25\ \text{ft})) + \left(3 \times (-0.043631\ \text{kip/ft}) \times (6.25\ \text{ft})^2\right)}{(6 \times (0.053662\ \text{kipft/ft})) + (4 \times (-0.043631\ \text{kip/ft}) \times (6.25\ \text{ft}))}$$

$$a = 4.5688\ \text{ft}$$

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

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

$$V_{max} = ((-0.043631\ \text{kip/ft}) \times (48\ \text{in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.5688\ \text{ft})}{(6.25\ \text{ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.5688\ \text{ft})}{(6.25\ \text{ft})}\right)^3\right]\right]$$

$$V_{max} = 0.17867\ \text{kip}$$

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

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

$$M_{max} = ((-0.043631\ \text{kip/ft}) \times (48\ \text{in}) \times (6.25\ \text{ft})) \times \left[\left(\frac{(1.2299\ \text{ft})}{(6.25\ \text{ft})} + \frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right) - \left[\left(\frac{4 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 3\right) \times \left(\frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^3\right] + \left[\left(\frac{3 \times (1.2299\ \text{ft})}{(6.25\ \text{ft})} + 2\right) \times \left(\frac{(4.5688\ \text{ft})}{2 \times (6.25\ \text{ft})}\right)^4\right]\right]$$

$$M_{max} = 0.46205\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.109\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2))\right]$$

$$A_{st,required} = -84.36\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[(-84.36\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))\right]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.109\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0026574$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.109 \text{ kip} \rightarrow 7109 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7109 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.43 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.43 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.43 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.43 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.71 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.301 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(10.301\ \text{kip})}{(110.71\ \text{kip})}$$

$$Ratio = 0.09304$$

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

Considering z-direction:

$V_{max} = 0.17867\ \text{kip}$ - Maximum shear force in the z-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(0.17867\ \text{kip})}{(110.71\ \text{kip})}$$

$$Ratio = 0.0016138$$

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:

$M_{max} = 30.526\ \text{kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(30.526\ \text{kipft})}{(249.6\ \text{kipft})}$$

$$Ratio = 0.1223$$

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

Considering z-direction:

$M_{max} = 0.46205\ \text{kipft}$ - Maximum moment in the z-direction,
$Ratio$ - Capacity

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

	$$Ratio = \frac{(0.46205\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.0018512$$	Status: **PASS** Ratio: **0.000**

REFERENCES	CALCULATIONS	RESULTS

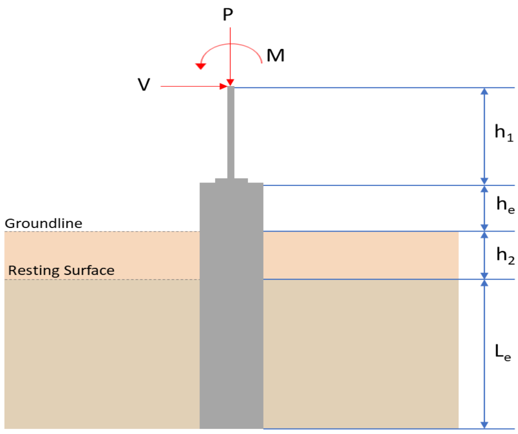
SkyCiv Foundation Design

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
$L = 6.75$ 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)	5.538	8.778
V_x (kip)	-3.459	-5.764
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	34.571	57.835

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.459\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.5508\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.571\ \mathrm{kipft}) + ((-3.459\ \mathrm{kip}) \times (0\ \mathrm{ft}))}{1.57 \times (48\ \mathrm{in})}$$

$$M_o = 5.5049\ \mathrm{kipft/ft}$$

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:
$L_{e,x} = 6.1809\ \mathrm{ft}$ - 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,

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

$$L_{e,req} = MAX[(6.1809\ \mathrm{ft}), (0\ \mathrm{ft})]$$

$$L_{e,req} = 6.181\ \mathrm{ft}$$

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

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

$$Ratio = \frac{(6.181\ \mathrm{ft})}{(6.75\ \mathrm{ft})}$$

$$Ratio = 0.9157$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b\,D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.538\ \mathrm{kip})}{(16\ \mathrm{ft}^2)}$$

$$q = 0.34613\ \mathrm{kip/ft}^2$$

Check bearing capacity ratio:

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.34613\ \mathrm{kip/ft}^2)}{(2000\ \mathrm{psf})}$$

$$Ratio = 0.17306$$

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

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:

$H_o = -0.5508 \text{ kip/ft}$ - Lateral force per length of pile,
$M_o = 5.5049 \text{ kipft/ft}$ - Overturning moment per length of pile,
a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5049 \text{ kipft/ft}) \times (6.75 \text{ ft})) + \left(3 \times (-0.5508 \text{ kip/ft}) \times (6.75 \text{ ft})^2\right)}{(6 \times (5.5049 \text{ kipft/ft})) + (4 \times (-0.5508 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6746 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.5049 \text{ kipft/ft})) + (3 \times (-0.5508 \text{ kip/ft}) \times (6.75 \text{ ft})]^2}{(6.75 \text{ ft})^2 \times [(3 \times (5.5049 \text{ kipft/ft})) + (2 \times (-0.5508 \text{ kip/ft}) \times (6.75 \text{ ft})]}$$

$$p = 0.21407 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e,

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

$$s = \frac{6 \times [(2 \times (5.5049 \text{ kipft/ft})) + ((-0.5508 \text{ kip/ft}) \times (6.75 \text{ ft})]}{(6.75 \text{ ft})^2}$$

$$s = 0.96026 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6746 \text{ ft})}{2}$$

$$p_a = 0.3506 \text{ kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.21407 \text{ kip/ft}^2\right)}{\left(0.3506 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.61059$$

Status: **PASS**
Ratio: **0.610**

p_s - Allowable lateral soil pressure at depth L_e,

$$p_s = R\, L_e$$

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

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

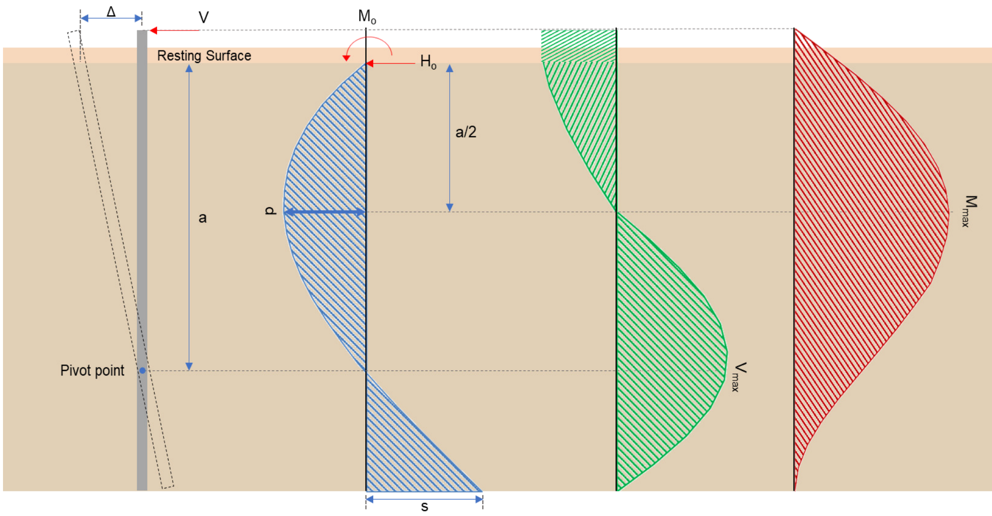
$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{\left(0.96026 \text{ kip/ft}^2\right)}{\left(1.0125 \text{ kip/ft}^2\right)}$$

$$Ratio = 0.94841$$

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



Pile Rotation | Pressure Diagram | Shear Diagram | Moment Diagram

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.764\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.91783\text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.835\text{ kipft}) + ((-5.764\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 9.2094\text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.2094\text{ kipft/ft})}{(-0.91783\text{ kip/ft})}$$

$$E = 10.034\text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.2094\text{ kipft/ft}) \times (6.75\text{ ft})) + \left(3 \times (-0.91783\text{ kip/ft}) \times (6.75\text{ ft})^2\right)}{(6 \times (9.2094\text{ kipft/ft})) + (4 \times (-0.91783\text{ kip/ft}) \times (6.75\text{ ft}))}$$

$$a = 4.6742\text{ ft}$$

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

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

$$V_{max} = ((-0.91783\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.034\text{ ft})}{(6.75\text{ ft})} + 3\right) \times \left(\frac{(4.6742\text{ ft})}{(6.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.034\text{ ft})}{(6.75\text{ ft})} + 2\right) \times \left(\frac{(4.6742\text{ ft})}{(6.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 12.078\text{ kip}$$

$$V_{max} = 12.078\ \text{kip}$$

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

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

$$M_{max} = ((-0.91783\ \text{kip/ft}) \times (48\ \text{in}) \times (6.75\ \text{ft})) \times \left[\left(\frac{(10.034\ \text{ft})}{(6.75\ \text{ft})} + \frac{(4.6742\ \text{ft})}{2 \times (6.75\ \text{ft})} \right) - \left[\left(\frac{4 \times (10.034\ \text{ft})}{(6.75\ \text{ft})} + 3 \right) \times \left(\frac{(4.6742\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.034\ \text{ft})}{(6.75\ \text{ft})} + 2 \right) \times \left(\frac{(4.6742\ \text{ft})}{2 \times (6.75\ \text{ft})} \right)^4 \right] \right]$$

$$M_{max} = 38.517\ \text{kipft}$$

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 $A_{st,required}$

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

$$A_{st,required} = Min \left[\frac{\frac{(8.778\ \text{kip})}{(0.65)\times(0.8)} - (0.85 \times (2.5\ \text{ksi}) \times (2304\ \text{in}^2))}{(60\ \text{ksi}) - (0.85 \times (2.5\ \text{ksi}))}, (0.08 \times (2304\ \text{in}^2)) \right]$$

$$A_{st,required} = -84.304\ \text{in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max\,[(-84.304\ \text{in}^2), (0.0018 \times (2304\ \text{in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

Status: PASS
Ratio: 0.970

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.778 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0032813$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.778 \text{ kip} \rightarrow 8778 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8778 \text{ lbf})}{6 \times (2304 \text{ in}^2)}\right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.66 \text{ kip}$$

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

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

$$V_c = Min\left[(296.21 \text{ kip}), (119.66 \text{ kip}), (348.89 \text{ kip})\right]$$

$$V_c = 119.66 \text{ kip}$$

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.66 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.86 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.078 \text{ kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(12.078 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.10895$$

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

	Flexural Strength (ACI 318-19, LRFD) S_m - Section modulus $$S_m = \frac{b\,D^2}{6}$$ $$S_m = \frac{(48\text{ in}) \times (48\text{ in})^2}{6}$$ $$S_m = 18432\text{ in}^3$$ $\lambda = 1$ - 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:** $M_{max} = 38.517\text{ kipft}$ - Maximum moment in the x-direction, $Ratio$ - Capacity $$Ratio = \frac{M_{max}}{\phi M_n}$$ $$Ratio = \frac{(38.517\text{ kipft})}{(249.6\text{ kipft})}$$ $$Ratio = 0.15431$$	Status: **PASS** Ratio: **0.150**