

Project Name: Coleman Residence- 4566 Casa Redonda **Date:** Tue Aug 19 2025

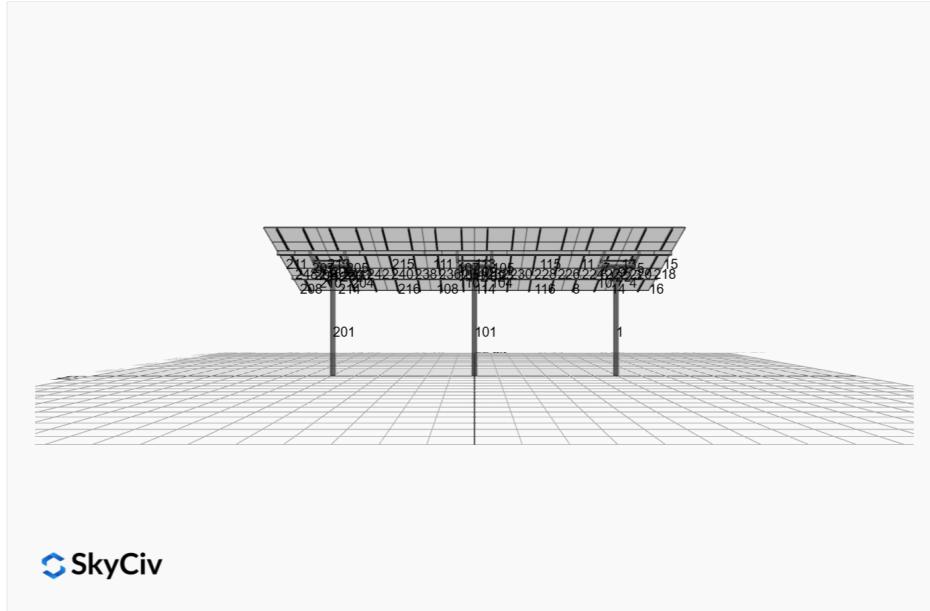
Location: 4566 Casa Redonda Dr, Shingle Springs, CA **Number of Modules:** 40
95682, USA

Unique ID: 3P-17-6TOP-SD-24-L-5Hx8W-E9C9

Number of Poles: 3

Date Sold:

Dealer: _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	45.87 ft
Winter Tilt Angle	20
Front Edge Clearance	10 ft

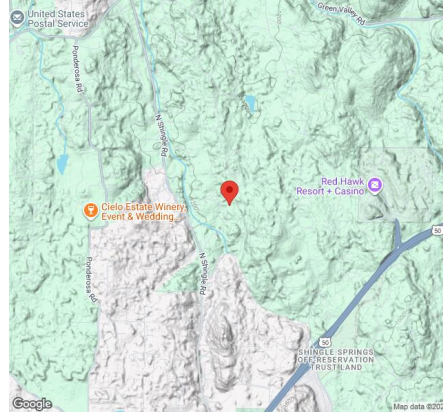
MT Solar Bill of Materials (3P-17-6TOP-SD-24-L-5Hx8W-E9C9)

Part	Short Description	BOM Qty
MTS-PC-6	6IN 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-ANGLE-4PK	Angle Clamp	8

Rail Bill of Materials

Part	Qty
Rails (226in)	16
Rail Attachment	64
Module Mid Clamp	64
Module End Clamp	32
Ground Lug	8

Site Details:



Site Address: 4566 Casa Redonda Dr, Shingle Springs, CA 95682, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	67.80in
Number of Rows:	5
Number of Columns:	8
Total Number of Modules:	40
Winter Tilt Angle:	20
Front Edge Clearance:	10
Total Array Height at Tilt:	16.43 ft
Total Frame Length:	45.50 ft
Module Info/Notes:	JA Solar JAM54S31-405/MR
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	45.87 ft
Rail Length:	225.75 in
Rail Spacing:	2.87 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	13.22 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: 5.25 ft Pile 2: 5.25 ft Pile 3: 5.25 ft
Foundation Volume:	9.333 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	4566 Casa Redonda Dr, Shingle Springs, CA 95682, USA
Wind Speed:	89 mph

Snow Load:

20 psf

Design Disclaimer

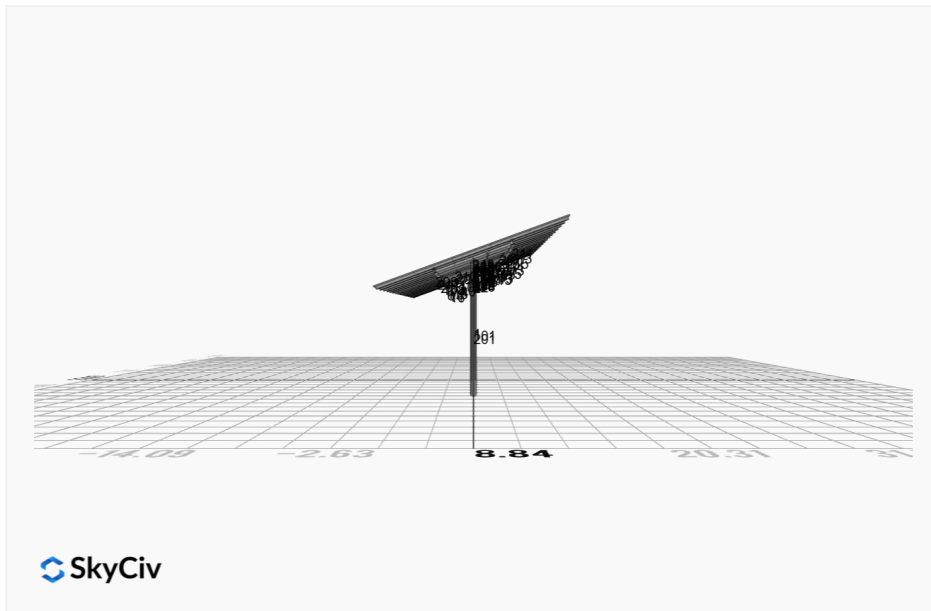
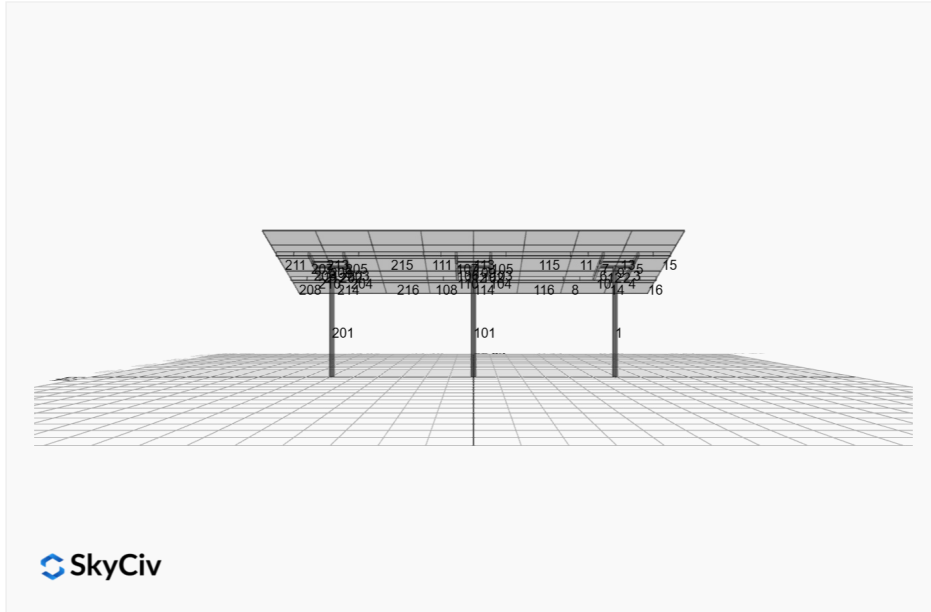
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

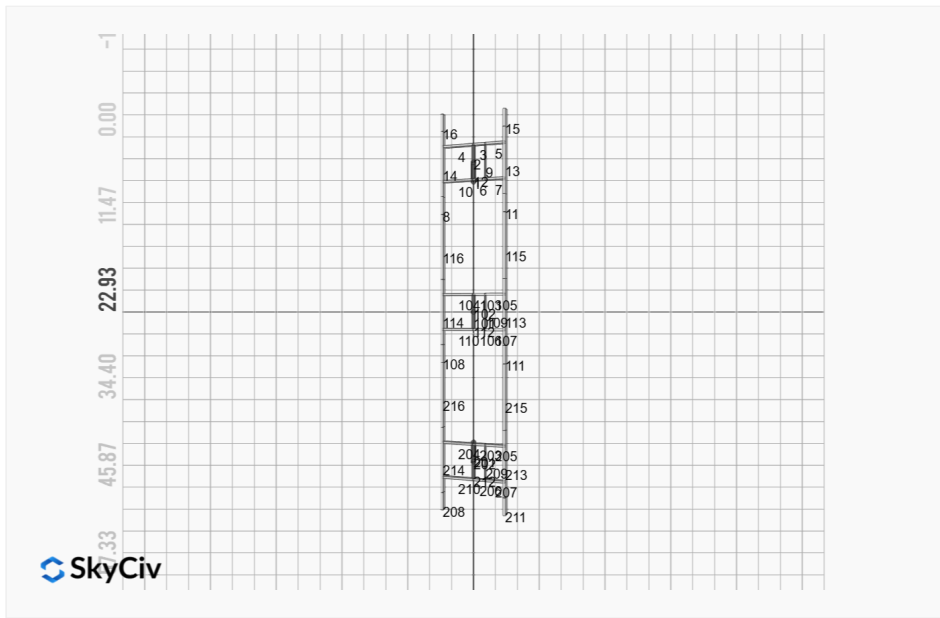
AutoDesigner Input

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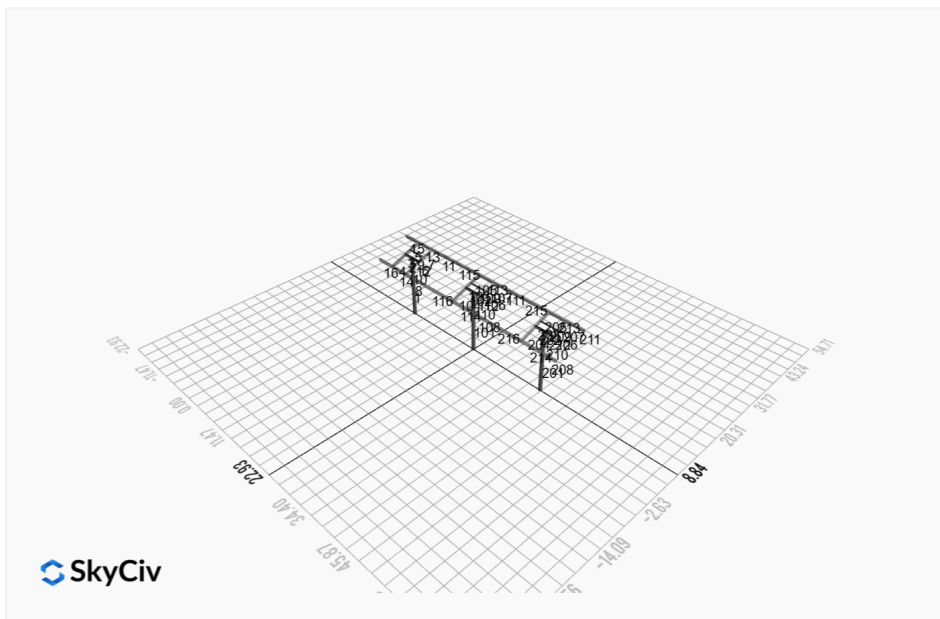
Design Notes:

- 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

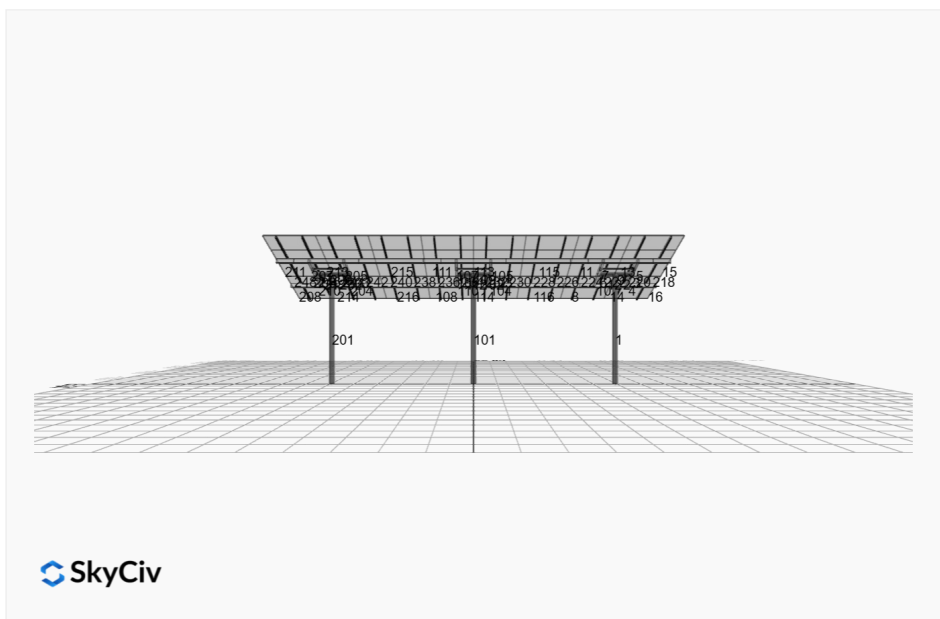




SkyCiv

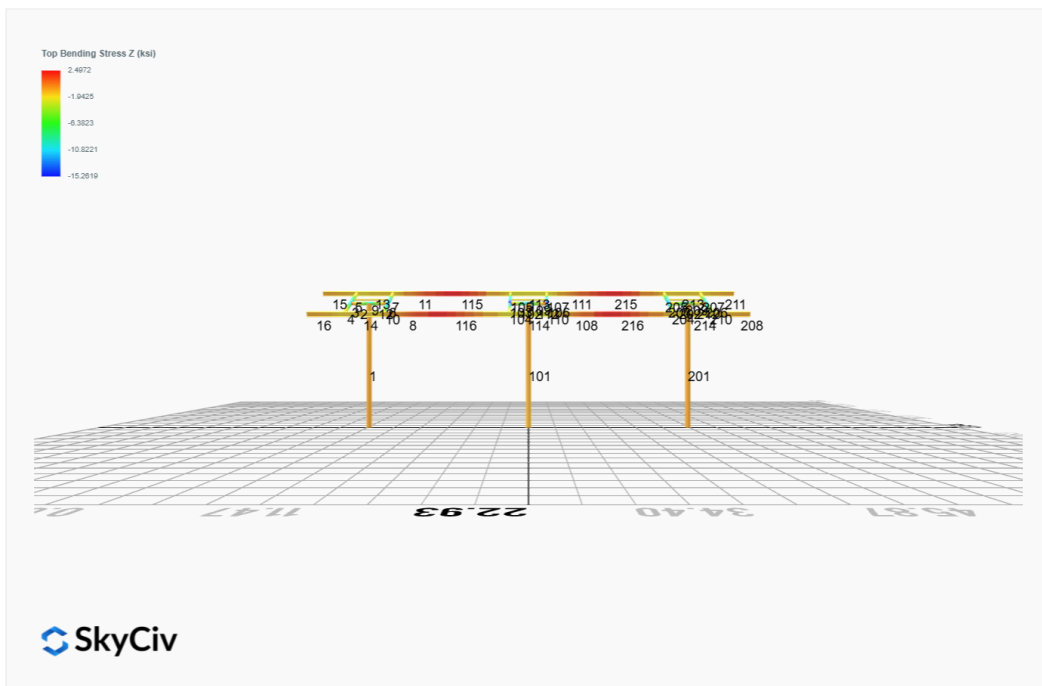
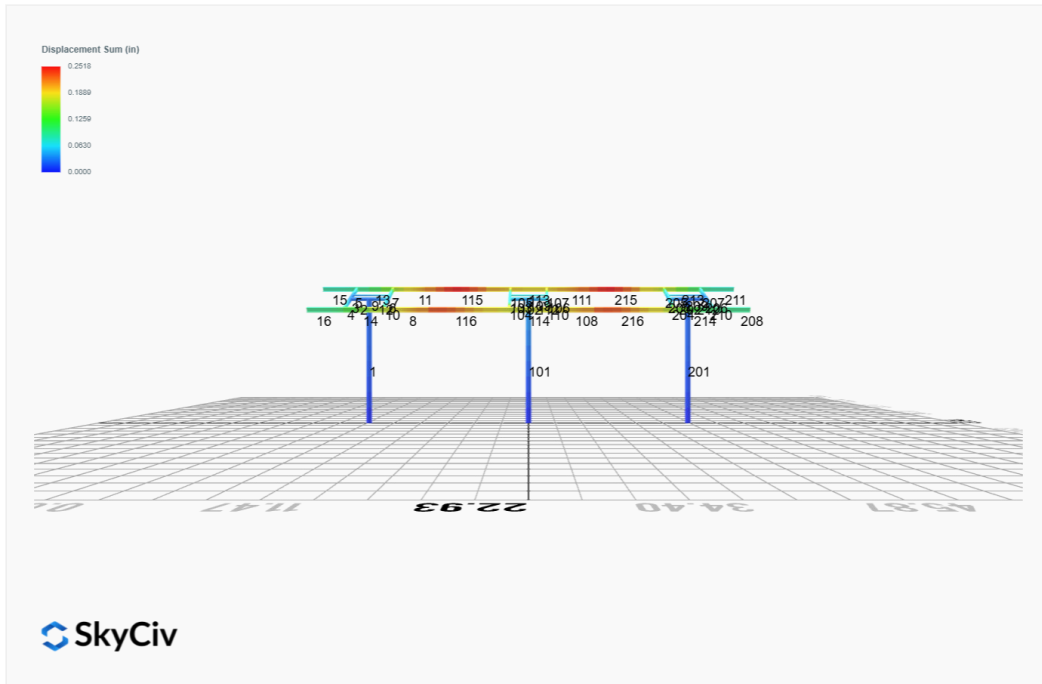


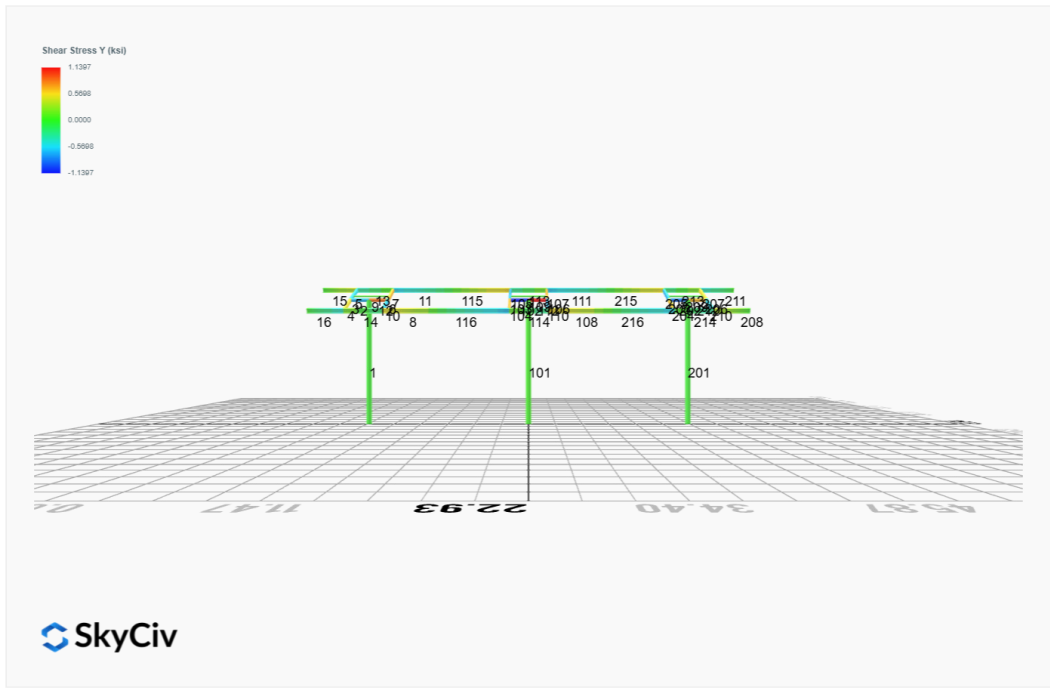
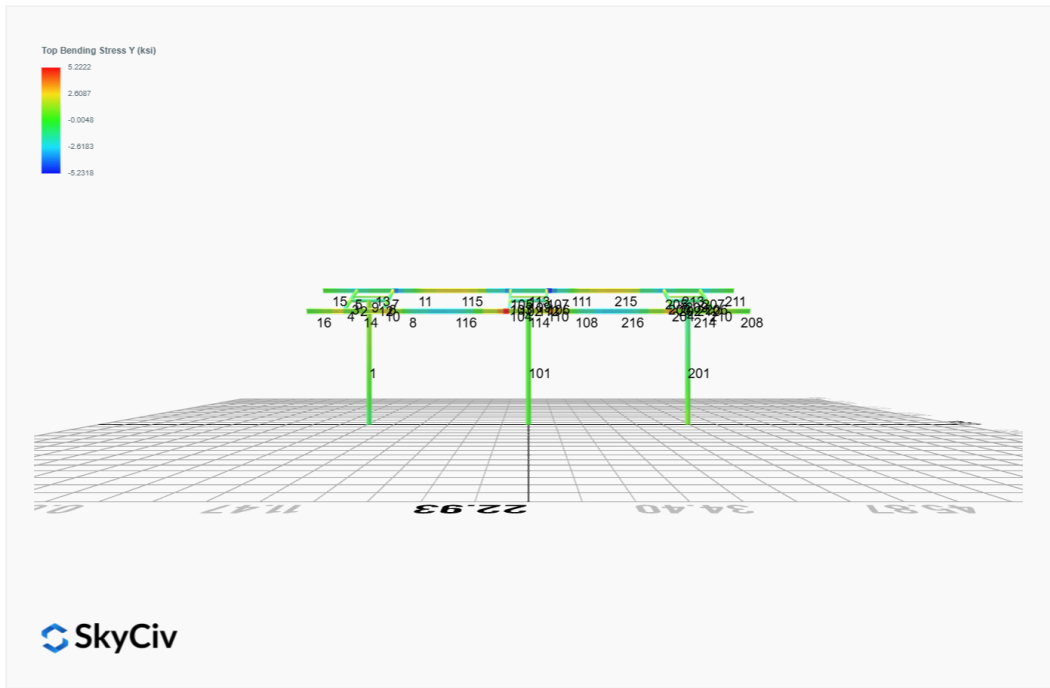
SkyCiv

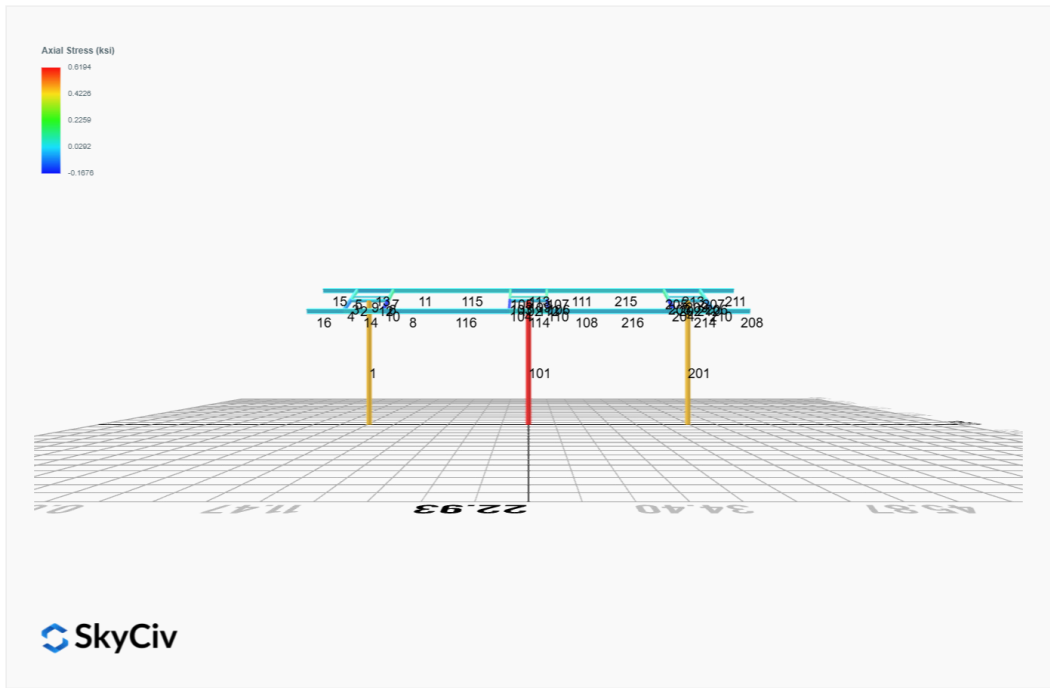


SkyCiv

FEM Results (Envelope Worst Case for each member)







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	0.0069	1.8495	0.0401	0.1703	-0.0217	-0.0523
ULS: 2. D + L	0.0069	1.8495	0.0401	0.1703	-0.0217	-0.0523
ULS: 3. D + (S or Lr or R)	0.0196	4.5435	0.1149	0.4886	-0.0625	-0.1802
ULS: 3. D + (S or Lr or R)	0.0069	1.8495	0.0401	0.1703	-0.0217	-0.0523
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0165	3.8700	0.0962	0.4091	-0.0523	-0.1482
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0069	1.8495	0.0401	0.1703	-0.0217	-0.0523
ULS: 5b. D + 0.7E	0.0069	1.8495	0.0401	0.1703	-0.0217	-0.0523
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0165	3.8700	0.0962	0.4091	-0.0523	-0.1482
ULS: 8. 0.6D + 0.7E	0.0041	1.1097	0.0241	0.1022	-0.0130	-0.0314
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7483	3.8161	0.1081	0.4560	-0.1176	10.8055
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7483	3.8161	0.1081	0.4560	-0.1176	10.8055
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6408	0.1828	-0.0149	-0.0596	0.0539	-7.7909
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5789	0.3955	-0.0150	-0.0597	0.0610	-12.3130
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5499	5.3449	0.1472	0.6233	-0.1242	7.9952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5499	5.3449	0.1472	0.6233	-0.1242	7.9952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4919	2.6199	0.0549	0.2366	0.0044	-5.9521
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4455	2.7795	0.0548	0.2366	0.0097	-9.3437
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5595	3.3245	0.0911	0.3846	-0.0937	8.0911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5595	3.3245	0.0911	0.3846	-0.0937	8.0911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4823	0.5994	-0.0012	-0.0021	0.0350	-5.8562
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4359	0.7590	-0.0013	-0.0022	0.0403	-9.2478
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7510	3.0763	0.0921	0.3878	-0.1090	10.8265
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7510	3.0763	0.0921	0.3878	-0.1090	10.8265
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6380	-0.5570	-0.0310	-0.1277	0.0626	-7.7699
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5762	-0.3443	-0.0311	-0.1278	0.0696	-12.2920

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.1693
Shear X	-1.2586
Shear Z	0.2265
Moment X	0.9650
Moment Y (Twist)	0.2104
Moment Z	21.5594

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	5.3449
Shear X	-0.7510
Shear Z	0.1472
Moment X	0.6233
Moment Y (Twist)	0.1242
Moment Z	12.3130

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	-0.0138	2.2599	-0.0000	0.0000	0.0000	0.1707
ULS: 2. D + L	-0.0138	2.2599	-0.0000	0.0000	0.0000	0.1707
ULS: 3. D + (S or Lr or R)	-0.0393	5.7169	0.0000	0.0000	0.0000	0.4610
ULS: 3. D + (S or Lr or R)	-0.0138	2.2599	-0.0000	0.0000	0.0000	0.1707
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0329	4.8526	-0.0000	0.0000	0.0000	0.3884

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0138	2.2599	-0.0000	0.0000	0.0000	0.1707
ULS: 5b. D + 0.7E	-0.0138	2.2599	-0.0000	0.0000	0.0000	0.1707
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0329	4.8526	-0.0000	0.0000	0.0000	0.3884
ULS: 8. 0.6D + 0.7E	-0.0083	1.3559	-0.0000	0.0000	0.0000	0.1024
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8543	4.7854	-0.0000	0.0000	0.0000	12.1082
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8543	4.7854	-0.0000	0.0000	0.0000	12.1082
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7131	0.1133	-0.0000	0.0000	0.0000	-8.4250
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5755	0.4053	-0.0000	0.0000	0.0000	-12.8660
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6633	6.7468	-0.0000	0.0000	0.0000	9.3415
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6633	6.7468	-0.0000	0.0000	0.0000	9.3415
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5123	3.2427	-0.0000	0.0000	0.0000	-6.0584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4091	3.4617	-0.0000	0.0000	0.0000	-9.3891
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6442	4.1540	-0.0000	0.0000	0.0000	9.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6442	4.1540	-0.0000	0.0000	0.0000	9.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5314	0.6499	-0.0000	0.0000	0.0000	-6.2761
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4282	0.8690	-0.0000	0.0000	0.0000	-9.6068
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8487	3.8815	-0.0000	0.0000	0.0000	12.0399
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8487	3.8815	-0.0000	0.0000	0.0000	12.0399
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7186	-0.7907	-0.0000	0.0000	0.0000	-8.4933
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5810	-0.4986	-0.0000	0.0000	0.0000	-12.9343

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.3462
Shear X	-1.4202
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	22.4465

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.7468
Shear X	-0.8543
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	12.9343

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	0.0069	1.8495	-0.0401	-0.1703	0.0217	-0.0523
ULS: 2. D + L	0.0069	1.8495	-0.0401	-0.1703	0.0217	-0.0523
ULS: 3. D + (S or Lr or R)	0.0196	4.5435	-0.1149	-0.4886	0.0625	-0.1802
ULS: 3. D + (S or Lr or R)	0.0069	1.8495	-0.0401	-0.1703	0.0217	-0.0523
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0165	3.8700	-0.0962	-0.4091	0.0523	-0.1482
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0069	1.8495	-0.0401	-0.1703	0.0217	-0.0523
ULS: 5b. D + 0.7E	0.0069	1.8495	-0.0401	-0.1703	0.0217	-0.0523
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0165	3.8700	-0.0962	-0.4091	0.0523	-0.1482
ULS: 8. 0.6D + 0.7E	0.0041	1.1097	-0.0241	-0.1022	0.0130	-0.0314
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.7483	3.8161	-0.1081	-0.4560	0.1176	10.8056
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.7483	3.8161	-0.1081	-0.4560	0.1176	10.8056
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6408	0.1828	0.0149	0.0596	-0.0539	-7.7909
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5789	0.3955	0.0150	0.0597	-0.0610	-12.3130

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5499	5.3449	-0.1472	-0.6233	0.1242	7.9952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5499	5.3449	-0.1472	-0.6233	0.1242	7.9952
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4919	2.6199	-0.0549	-0.2366	-0.0044	-5.9521
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4455	2.7795	-0.0548	-0.2366	-0.0097	-9.3437
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5595	3.3245	-0.0911	-0.3846	0.0937	8.0911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5595	3.3245	-0.0911	-0.3846	0.0937	8.0911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4823	0.5994	0.0012	0.0021	-0.0350	-5.8562
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4359	0.7590	0.0013	0.0022	-0.0403	-9.2478
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.7510	3.0763	-0.0921	-0.3878	0.1090	10.8265
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.7510	3.0763	-0.0921	-0.3878	0.1090	10.8265
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6380	-0.5570	0.0310	0.1277	-0.0626	-7.7699
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5762	-0.3443	0.0311	0.1278	-0.0696	-12.2920

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.1693
Shear X	-1.2586
Shear Z	-0.2265
Moment X	-0.9649
Moment Y (Twist)	0.2103
Moment Z	21.5598

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	5.3449
Shear X	-0.7510
Shear Z	-0.1472
Moment X	-0.6233
Moment Y (Twist)	0.1242
Moment Z	12.3130

Project Details

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 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial

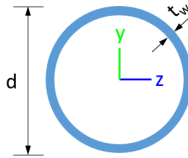


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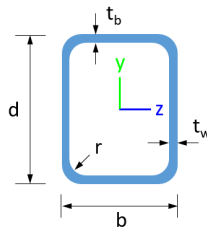
Design Factors			
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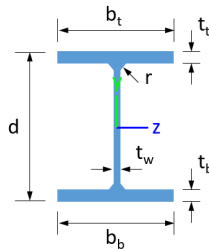
Section Dimensions



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4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



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)
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101	251.10	57.31	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.89	10.99	6.26	29.14	16.61
104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	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	72.84	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	18.20	6.45	30.09	45.74
114	120.60	84.03	17.99	6.45	30.09	45.74
115	120.60	84.26	18.66	6.45	30.09	45.74
116	120.60	84.26	18.82	6.45	30.09	45.74
201	251.16	57.31	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	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	72.84	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	20.53	6.45	30.09	45.74
214	120.60	84.03	19.69	6.45	30.09	45.74
215	120.60	84.26	18.97	6.45	30.09	45.74
216	120.60	84.26	19.01	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	0.143	0.510	0.048	0.017	0.003	0.525	#13	0.742	Not Required	Pass
2	0.001	0.356	0.077	0.082	0.013	0.415	#21	0.034	Not Required	Pass
3	0.006	0.589	0.039	0.058	0.009	0.631	#21	0.044	Not Required	Pass
4	0.005	0.570	0.068	0.058	0.014	0.641	#21	0.078	Not Required	Pass
5	0.006	0.365	0.047	0.059	0.008	0.370	#21	0.073	Not Required	Pass
6	0.008	0.721	0.093	0.074	0.021	0.816	#21	0.044	Not Required	Pass
7	0.009	0.447	0.121	0.072	0.023	0.473	#21	0.073	Not Required	Pass
8	0.001	0.115	0.064	0.038	0.008	0.179	#21	0.088	Not Required	Pass
9	0.002	0.072	0.049	0.002	0.002	0.121	#21	0.198	Not Required	Pass
10	0.009	0.686	0.107	0.069	0.019	0.742	#21	0.078	Not Required	Pass
11	0.002	0.116	0.062	0.040	0.008	0.179	#21	0.088	Not Required	Pass
12	0.001	0.481	0.090	0.102	0.016	0.551	#21	0.052	Not Required	Pass
13	0.003	0.114	0.158	0.056	0.011	0.204	#21	0.265	Not Required	Pass
14	0.003	0.110	0.156	0.053	0.011	0.213	#24	0.177	Not Required	Pass
15	0.000	0.023	0.023	0.018	0.003	0.046	#21	Not Required	Not Required	Pass

16	0.000	0.023	0.023	0.018	0.003	0.046	#21	Not Required	Not Required	Pass
101	0.181	0.531	0.000	0.019	0.000	0.576	#13	0.742	Not Required	Pass
102	0.001	0.538	0.103	0.117	0.017	0.624	#21	0.052	Not Required	Pass
103	0.009	0.825	0.073	0.083	0.015	0.900	#21	0.044	Not Required	Pass
104	0.009	0.817	0.134	0.082	0.023	0.918	#21	0.078	Not Required	Pass
105	0.009	0.512	0.138	0.082	0.027	0.546	#21	0.073	Not Required	Pass
106	0.009	0.825	0.073	0.083	0.015	0.900	#21	0.044	Not Required	Pass
107	0.009	0.512	0.138	0.082	0.027	0.546	#21	0.073	Not Required	Pass
108	0.001	0.086	0.064	0.046	0.008	0.121	#21	0.088	Not Required	Pass
109	0.006	0.075	0.035	0.001	0.000	0.113	#21	0.198	Not Required	Pass
110	0.009	0.817	0.134	0.082	0.023	0.918	#21	0.078	Not Required	Pass
111	0.002	0.098	0.065	0.046	0.008	0.133	#21	0.088	Not Required	Pass
112	0.001	0.538	0.103	0.117	0.017	0.624	#21	0.052	Not Required	Pass
113	0.003	0.168	0.187	0.061	0.011	0.337	#21	0.265	Not Required	Pass
114	0.005	0.191	0.187	0.061	0.011	0.355	#21	0.265	Not Required	Pass
115	0.003	0.174	0.091	0.046	0.008	0.265	#21	0.321	Not Required	Pass
116	0.002	0.163	0.092	0.046	0.008	0.255	#21	0.321	Not Required	Pass
201	0.143	0.510	0.048	0.017	0.003	0.525	#13	0.742	Not Required	Pass
202	0.001	0.481	0.090	0.102	0.016	0.551	#21	0.052	Not Required	Pass
203	0.008	0.721	0.093	0.074	0.021	0.816	#21	0.044	Not Required	Pass
204	0.009	0.686	0.107	0.069	0.019	0.742	#21	0.078	Not Required	Pass
205	0.009	0.447	0.121	0.072	0.023	0.473	#21	0.073	Not Required	Pass
206	0.006	0.589	0.039	0.058	0.009	0.631	#21	0.044	Not Required	Pass
207	0.006	0.365	0.047	0.059	0.008	0.370	#21	0.073	Not Required	Pass
208	0.000	0.023	0.023	0.018	0.003	0.046	#21	Not Required	Not Required	Pass
209	0.002	0.072	0.049	0.002	0.002	0.121	#21	0.198	Not Required	Pass
210	0.005	0.570	0.068	0.058	0.014	0.641	#21	0.078	Not Required	Pass
211	0.000	0.023	0.023	0.018	0.003	0.046	#21	Not Required	Not Required	Pass
212	0.001	0.356	0.077	0.082	0.013	0.415	#21	0.034	Not Required	Pass
213	0.003	0.114	0.158	0.056	0.011	0.204	#21	0.177	Not Required	Pass
214	0.003	0.110	0.156	0.053	0.011	0.213	#24	0.265	Not Required	Pass
215	0.003	0.182	0.091	0.040	0.008	0.274	#21	0.321	Not Required	Pass
216	0.002	0.174	0.092	0.038	0.008	0.267	#21	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/compression)

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
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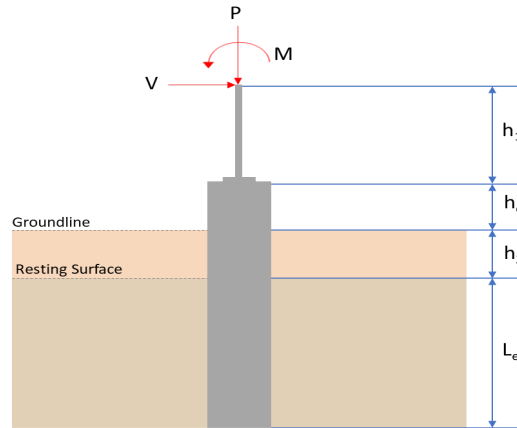
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ 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)	5.345	8.169
V_x (kip)	-0.751	-1.259
V_z (kip)	0.147	0.226
M_x (kipft)	0.623	0.965
M_z (kipft)	12.313	21.559

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.751 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11959 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(12.313 \text{ kipft}) + ((-0.751 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9607 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.9506 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.147 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.023408 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.623 \text{ kipft}) + ((0.147 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.099204 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2284 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9506 \text{ ft}), (2.2284 \text{ ft})]$$

$$L_{e,req} = 4.951 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.951 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.94305$$

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

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

$$q = \frac{(5.345 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33406 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33406 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16703$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.11959 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.9607 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9607 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.9607 \text{ kipft/ft})) + (4 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.577 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.9607 \text{ kipft/ft})) + (3 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (1.9607 \text{ kipft/ft})) + (2 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20887 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.9607 \text{ kipft/ft})) + ((-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.71695 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.577 \text{ ft})}{2}$$

$$p_a = 0.26827 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20887 \text{ kip/ft}^2)}{(0.26827 \text{ kip/ft}^2)}$$

$$Ratio = 0.77858$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.71695 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.91042$$

Status: **PASS**
Ratio: **0.780**

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

Considering z-direction:

$H_o = 0.023408 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.099204 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.099204 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.023408 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.099204 \text{ kipft/ft})) + (4 \times (0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6979 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.099204 \text{ kipft/ft})) + (3 \times (0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.099204 \text{ kipft/ft})) + (2 \times (0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.029343 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.099204 \text{ kipft/ft})) + ((0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.069942 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6979 \text{ ft})}{2}$$

$$p_a = 0.27734 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.029343 \text{ kip/ft}^2)}{(0.27734 \text{ kip/ft}^2)}$$

$$Ratio = 0.1058$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

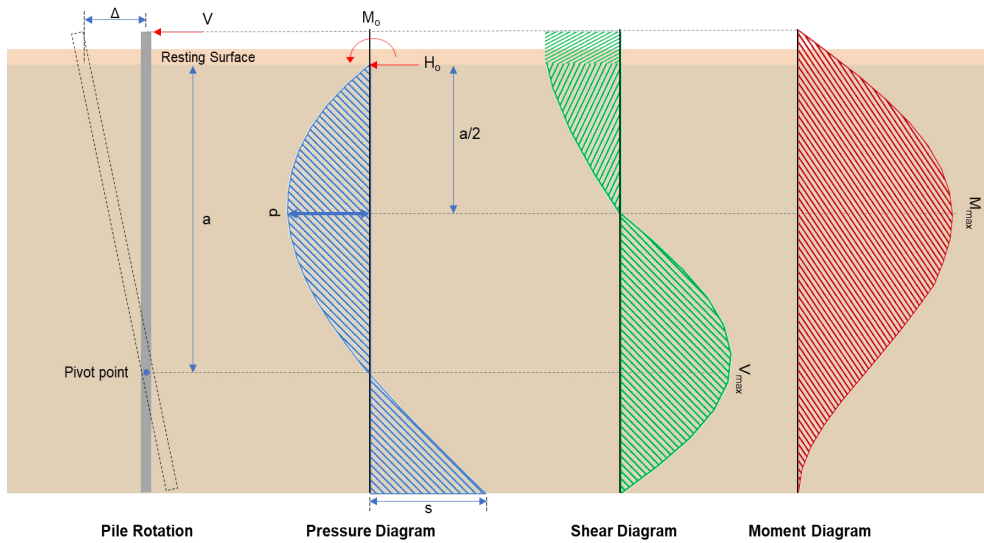
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.069942 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.088816$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.259 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.20048 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.559 \text{ kipft}) + ((-1.259 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.433 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.433 \text{ kipft/ft})}{(-0.20048 \text{ kip/ft})}$$

$$E = 17.124 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.433 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times 3.433) + (4 \times (-0.20048) \times 5.25)}$$

$$a = \frac{(6 \times (3.433 \text{ kipft/ft})) + (4 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft}))}{(6 \times (3.433 \text{ kipft/ft})) + (4 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5742 \text{ ft}$$

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

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

$$V_{max} = ((-0.20048 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.124 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5742 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.124 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5742 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.1625 \text{ kip}$$

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

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

$$M_{max} = ((-0.20048 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(17.124 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.124 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.124 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.166 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.226 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.035987 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.965 \text{ kipft}) + ((0.226 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15366 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15366 \text{ kipft/ft})}{(0.035987 \text{ kip/ft})}$$

$$E = 4.2699 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15366 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.035987 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.15366 \text{ kipft/ft})) + (4 \times (0.035987 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6971 \text{ ft}$$

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

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

$$V_{max} = ((0.035987 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6971 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6971 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.30244 \text{ kip}$$

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

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

$$M_{max} = ((0.035987 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(4.2699 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.72603 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.169 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.325 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.325 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.169 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0030536$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.169 \text{ kip} \rightarrow 8169 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8169 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{s,a}$ - Shear strength of steel (a)

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.1625 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.1625 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.046591$$

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

Considering z-direction:

$V_{max} = 0.30244 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.30244 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.0027295$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 13.166 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(13.166 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.05275$$

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

Considering z-direction:

$M_{max} = 0.72603 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.72603 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0029088$$

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

REFERENCES	CALCULATIONS	RESULTS
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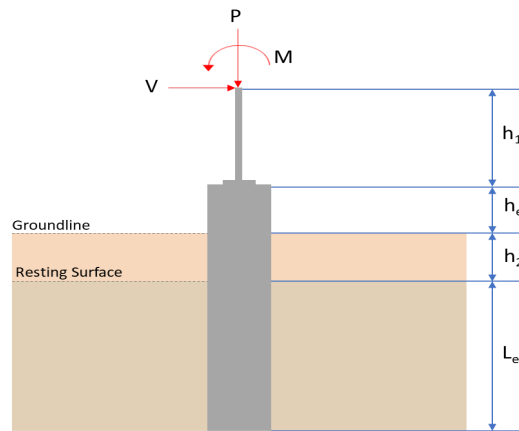
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ 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)	6.747	10.346
V_x (kip)	-0.854	-1.420
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	12.934	22.447

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.854 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13599 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(12.934 \text{ kipft}) + ((-0.854 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.0596 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.9876 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9876 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 4.988 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.988 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.9501$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.747 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.42169 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.42169 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.21084$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.13599 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.0596 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0596 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.13599 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.0596 \text{ kipft/ft})) + (4 \times (-0.13599 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5821 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.0596 \text{ kipft/ft})) + (3 \times (-0.13599 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.0596 \text{ kipft/ft})) + (2 \times (-0.13599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.21288 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.0596 \text{ kipft/ft})) + ((-0.13599 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74126 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5821 \text{ ft})}{2}$$

$$p_a = 0.26866 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21288 \text{ kip/ft}^2)}{(0.26866 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79236$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

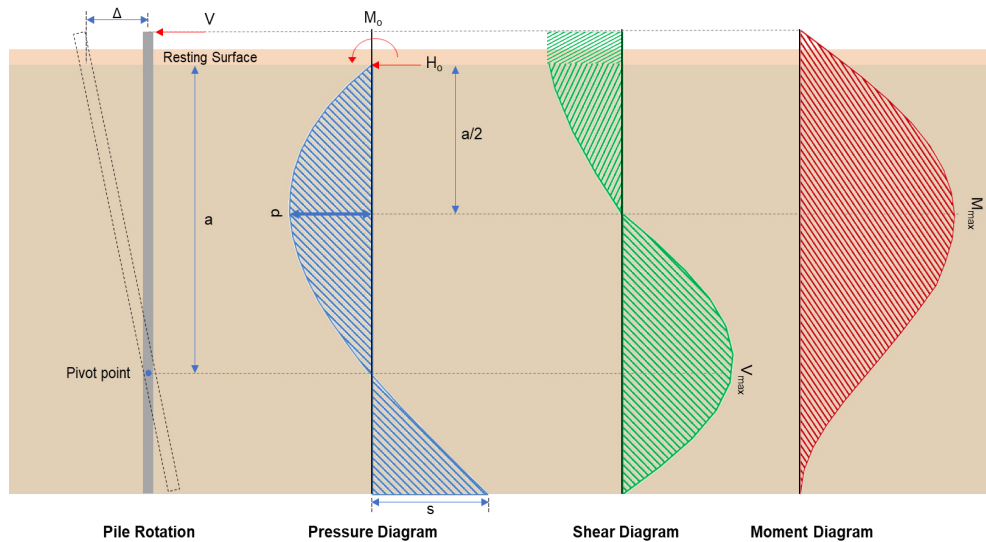
$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.74126 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.790**

**Shear force and Bending moment (x-direction, LRFD)** H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.42 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.22611 \text{ kip/ft}$$

 M_o - Moment per length of pile,

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

$$M_o = \frac{(22.447 \text{ kipft}) + ((-1.42 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.5744 \text{ kipft/ft}$$

 E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.5744 \text{ kipft/ft})}{(-0.22611 \text{ kip/ft})}$$

$$E = 15.808 \text{ ft}$$

 a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.5744 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.22611 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.5744 \text{ kipft/ft})) + (4 \times (-0.22611 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5793 \text{ ft}$$

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

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

$$V_{max} = ((-0.22611 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.808 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5793 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.808 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5793 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.4201 \text{ kip}$$

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

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

$$M_{max} = ((-0.22611 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(15.808 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5793 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.808 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5793 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (15.808 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5793 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.794 \text{ kipft}$$

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,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.346 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.252 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.252 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(10.346 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0038674$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 b_w = 48 in - Effective width,
 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.346 \text{ kip} \rightarrow 10346 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10346 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.86 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.86 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.86 \text{ kip}$$

22.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{s,a}$ - Shear strength of steel (a)

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.86 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.99 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.4201 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(5.4201 \text{ kip})}{(110.99 \text{ kip})}$$

$$\text{Ratio} = 0.048833$$

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

Flexural Strength (ACI 318-19, LFRD) S_m - Section modulus

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

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

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

 $\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

 M_n shall be the lesser of: $\phi M_{n,1}$

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

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

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

14.5.2.1b

 $\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction: $M_{max} = 13.794 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.794 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.055264$$

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

REFERENCES	CALCULATIONS	RESULTS
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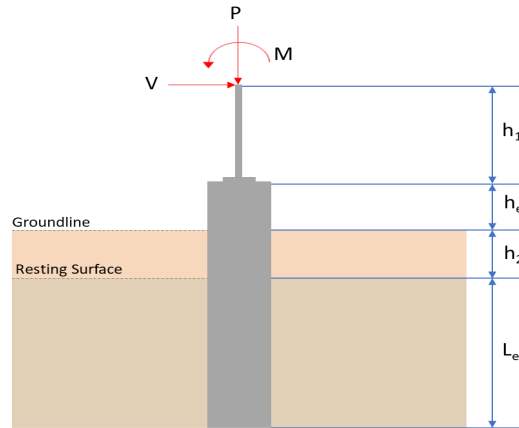
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ 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)	5.345	8.169
V_x (kip)	-0.751	-1.259
V_z (kip)	-0.147	-0.226
M_x (kipft)	-0.623	-0.965
M_z (kipft)	12.313	21.560

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.751 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.11959 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(12.313 \text{ kipft}) + ((-0.751 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9607 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.9506 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.147 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.023408 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.623 \text{ kipft}) + ((-0.147 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.099204 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7611 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9506 \text{ ft}), (1.7611 \text{ ft})]$$

$$L_{e,req} = 4.951 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.951 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.94305$$

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

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

$$q = \frac{(5.345 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33406 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33406 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16703$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.11959 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.9607 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9607 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.9607 \text{ kipft/ft})) + (4 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.577 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.9607 \text{ kipft/ft})) + (3 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (1.9607 \text{ kipft/ft})) + (2 \times (-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20887 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.9607 \text{ kipft/ft})) + ((-0.11959 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.71695 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.577 \text{ ft})}{2}$$

$$p_a = 0.26827 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20887 \text{ kip/ft}^2)}{(0.26827 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77858$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.71695 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91042$$

Status: **PASS**
Ratio: **0.780**

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

Considering z-direction:

$H_o = -0.023408 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.099204 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.099204 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.023408 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.099204 \text{ kipft/ft})) + (4 \times (-0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6979 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.099204 \text{ kipft/ft})) + (3 \times (-0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.099204 \text{ kipft/ft})) + (2 \times (-0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.00041586 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.099204 \text{ kipft/ft})) + ((-0.023408 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.016439 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6979 \text{ ft})}{2}$$

$$p_a = 0.27734 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00041586 \text{ kip/ft}^2)}{(0.27734 \text{ kip/ft}^2)}$$

$$Ratio = 0.0014995$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

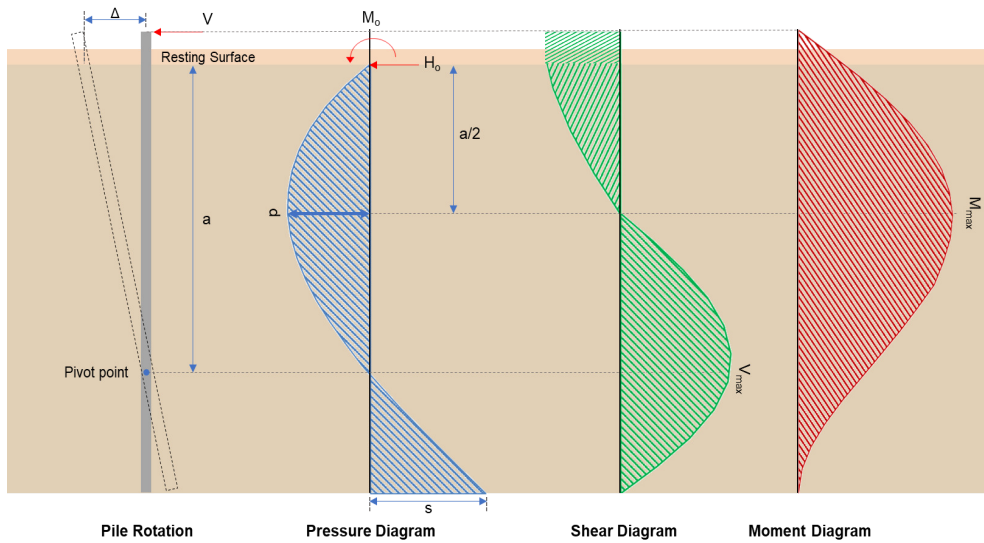
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.016439 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.020875$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.259 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.20048 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.56 \text{ kipft}) + ((-1.259 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4331 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.4331 \text{ kipft/ft})}{(-0.20048 \text{ kip/ft})}$$

$$E = 17.125 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4331 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.4331 \text{ kipft/ft})) + (4 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft})}{(6 \times (3.4331 \text{ kip/ft})) + (4 \times (-0.20048 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5742 \text{ ft}$$

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

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

$$V_{max} = ((-0.20048 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.125 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5742 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.125 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5742 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.1627 \text{ kip}$$

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

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

$$M_{max} = ((-0.20048 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(17.125 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.125 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.125 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5742 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.167 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.226 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.035987 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.965 \text{ kipft}) + ((-0.226 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15366 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.15366 \text{ kipft/ft})}{(-0.035987 \text{ kip/ft})}$$

$$E = 4.2699 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15366 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.035987 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.15366 \text{ kipft/ft})) + (4 \times (-0.035987 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6971 \text{ ft}$$

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

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

$$V_{max} = ((-0.035987 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6971 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6971 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.30244 \text{ kip}$$

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

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

$$M_{max} = ((-0.035987 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(4.2699 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2699 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6971 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.72603 \text{ kipft}$$

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,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

Required reinforcement due to axial load, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.169 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.325 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.325 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.169 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0030536$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.169 \text{ kip} \rightarrow 8169 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8169 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.57 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.57 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.57 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{s,a}$ - Shear strength of steel (a)

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.57 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.8 \text{ kip}$$

Considering x-direction:

$V_{max} = 5.1627 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(5.1627 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.046593$$

Considering z-direction:

$V_{max} = 0.30244 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.30244 \text{ kip})}{(110.8 \text{ kip})}$$

$$Ratio = 0.0027295$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 13.167 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(13.167 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.052752$$

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

Considering z-direction:

$M_{max} = 0.72603 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.72603 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0029088$$

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