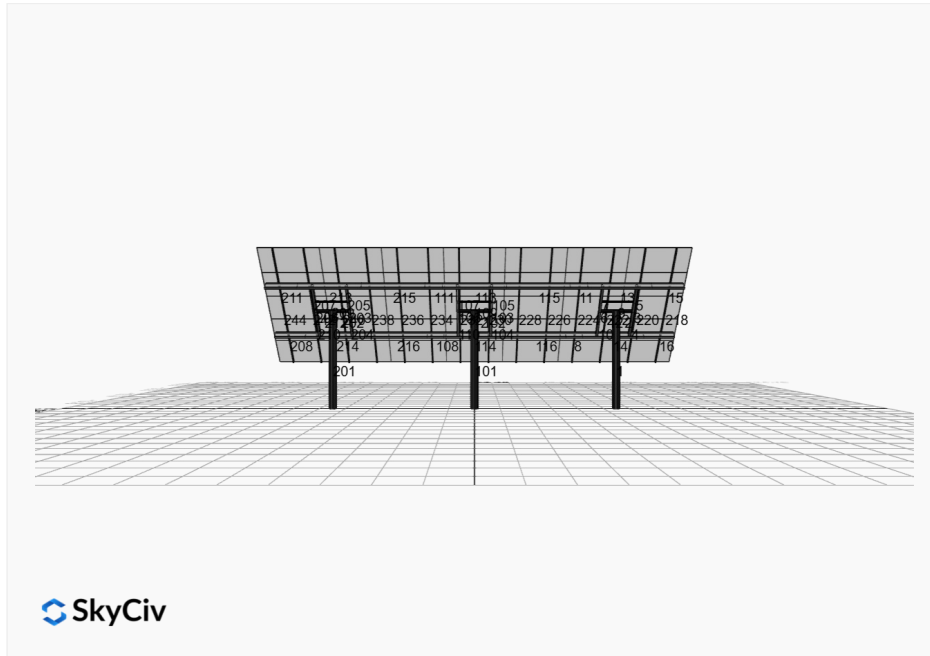


Project Name: test2 **Date:** Fri Oct 25 2024
Location: 28385 Post Office Rd, Mass City, MI 49948, **Number of Modules:** 35
 USA **Number of Poles:** 3
Unique ID: 3P-17-10TOP-XD-45-L-5Hx7W-HGJL **Date Sold:**
Dealer: _____



Array Dimensions N/S	17.42 ft
Array Dimensions E/W	49.53 ft
Winter Tilt Angle	50
Front Edge Clearance	5 ft

MT Solar Bill of Materials (3P-17-10TOP-XD-45-L-5Hx7W-HGJL)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	3
MTS-HF-XD	H-Frame Assembly-XD	3
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-57	57IN XD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	7

Rail Bill of Materials

Part	Qty
Rails (207in)	14
Rail Attachment	56
Module Mid Clamp	56
Module End Clamp	28
Ground Lug	7

Site Details:



Site Address: 28385 Post Office Rd, Mass City, MI 49948, USA

Array Specification

Duty Classification:	XD
Module Width:	41.30 in
Module Length:	83.90in
Number of Rows:	5
Number of Columns:	7
Total Number of Modules:	35
Winter Tilt Angle:	50
Front Edge Clearance:	5
Total Array Height at Tilt:	18.34 ft
Total Frame Length:	49.00 ft
Frame Weight:	4098 lbs
Array Dimensions N/S:	17.42 ft
Array Dimensions E/W:	49.53 ft
Rail Length:	209.00 in
Rail Spacing:	3.54 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	11.67 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.75 ft Pile 2: 6.75 ft Pile 3: 6.75 ft
Foundation Volume:	12.000 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	28385 Post Office Rd, Mass City, MI 49948, USA
Wind Speed:	98 mph
Snow Load:	70 psf

Design Disclaimer

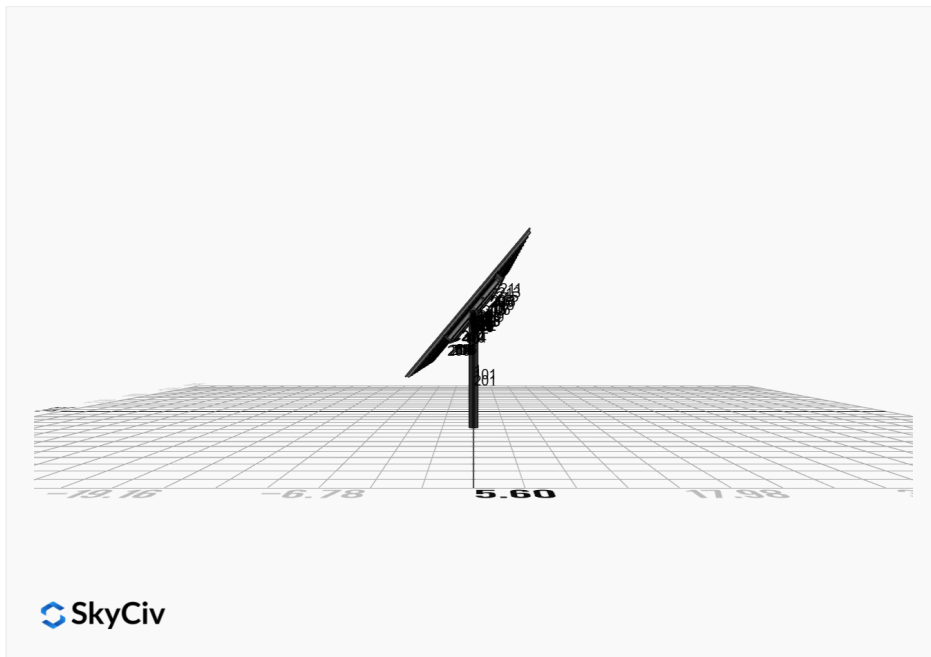
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

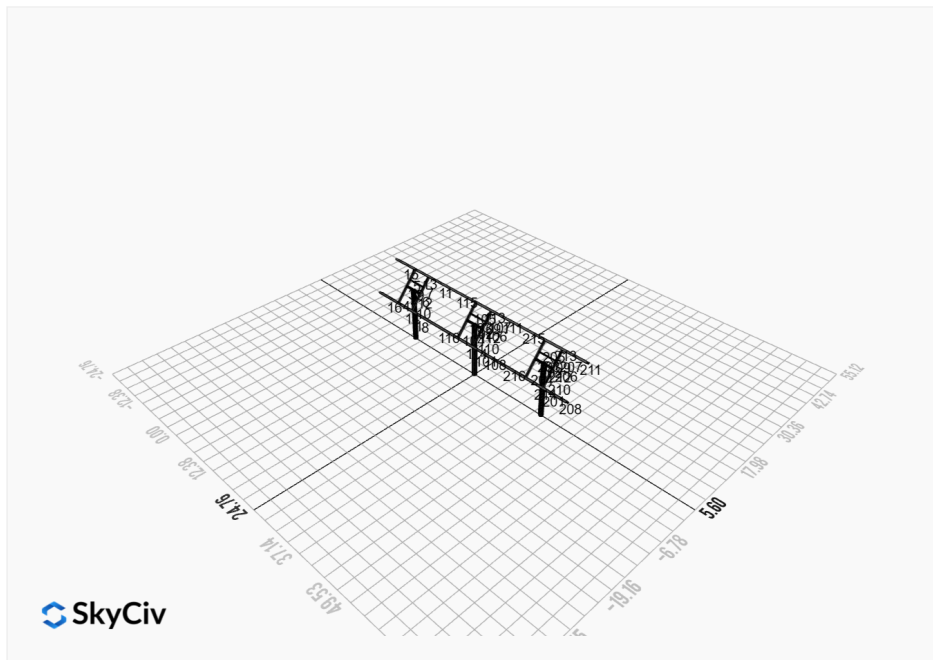
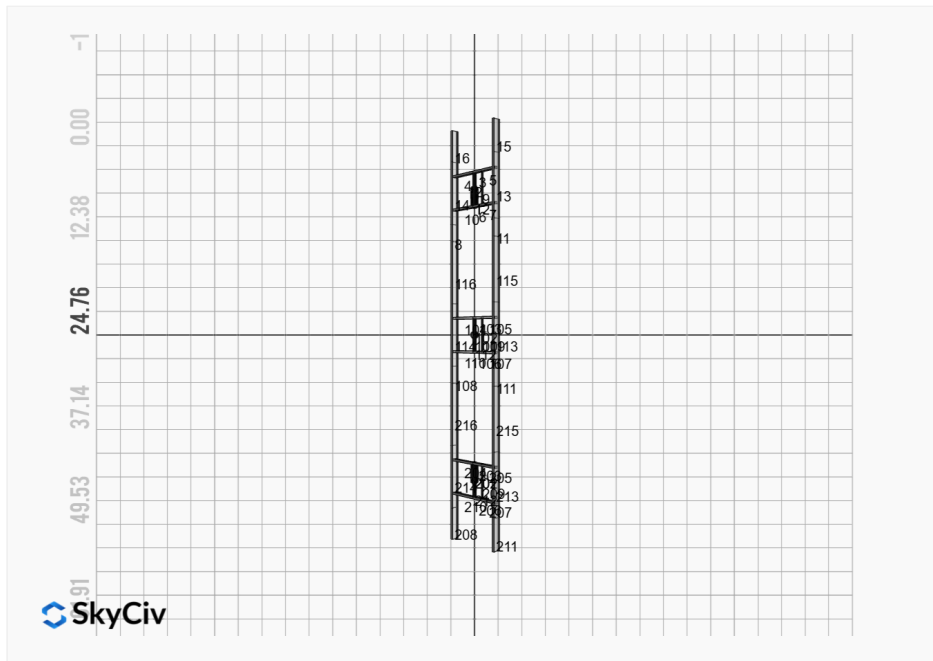
AutoDesigner Input

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Design Notes:

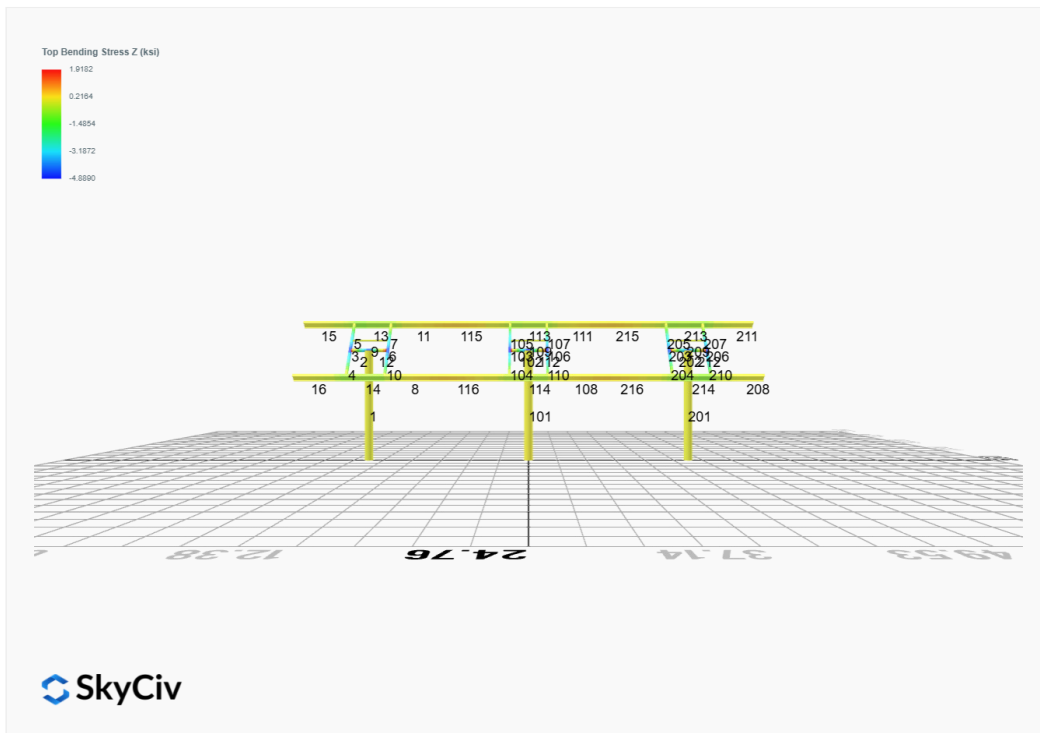
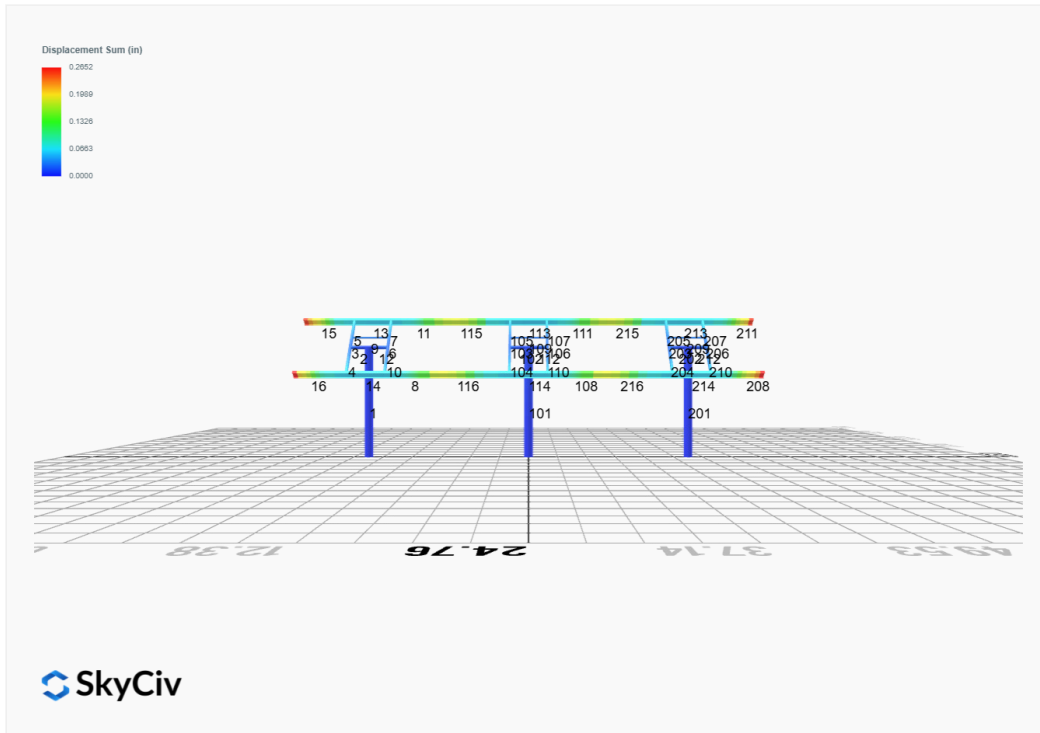
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

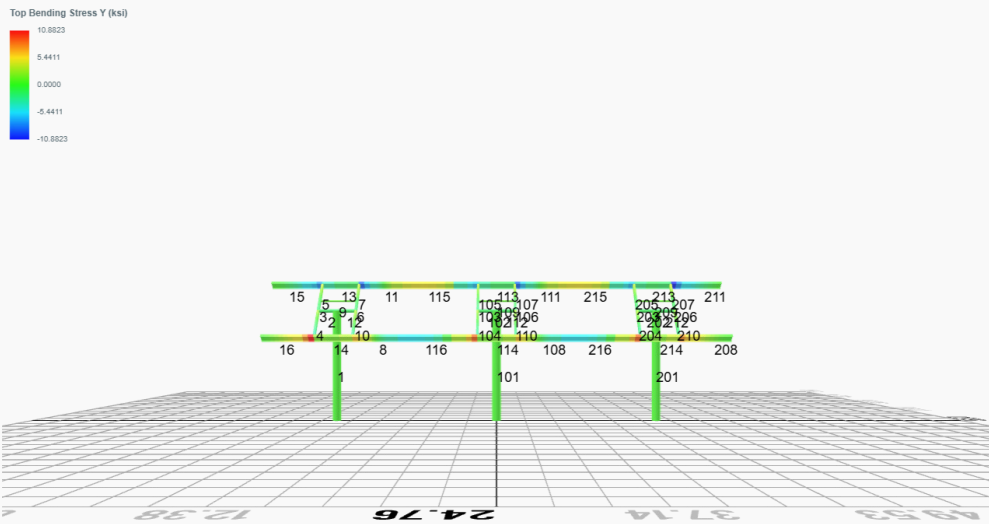




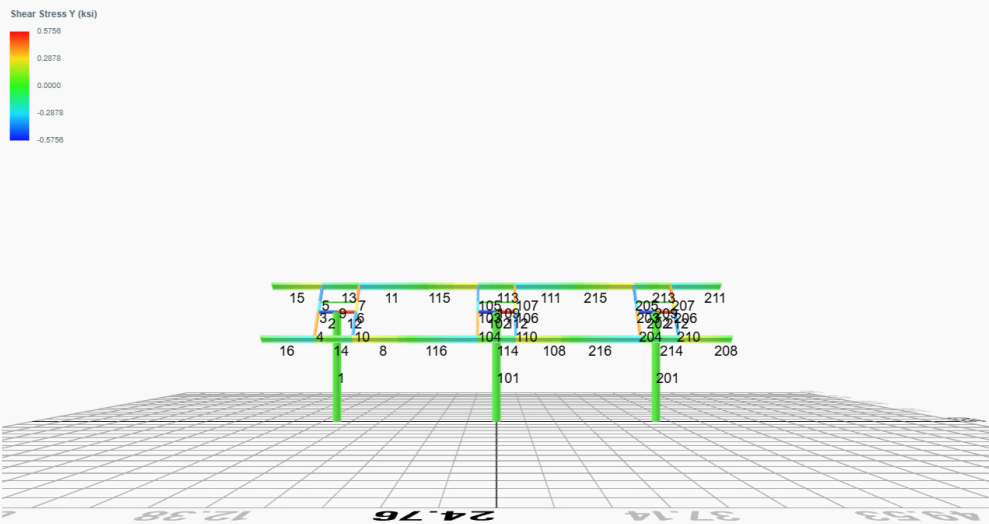


FEM Results (Envelope Worst Case for each member)

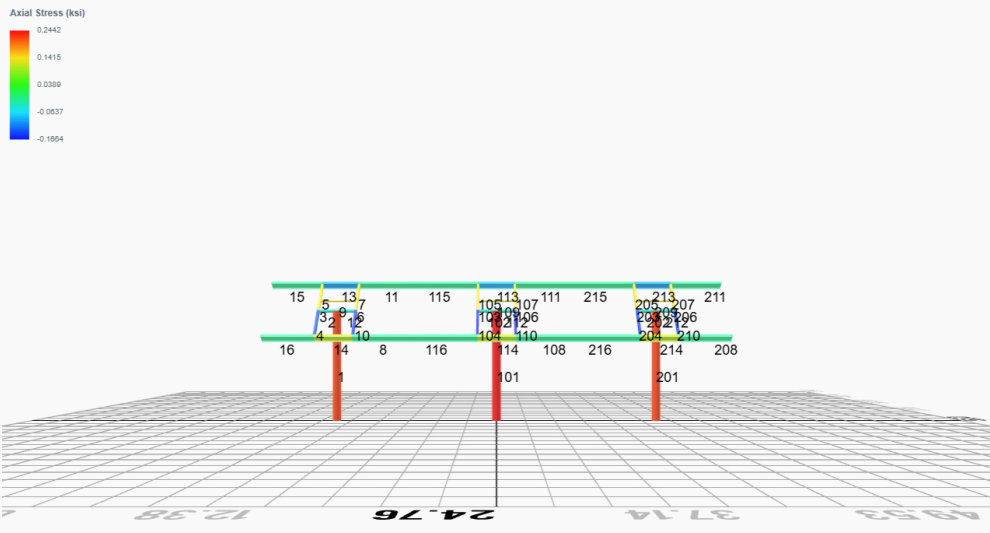




SkyCiv



SkyCiv



Reaction Forces for Foundation 1 (Node ID#1), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 2. D + L	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 3. D + (S or Lr or R)	-0.0187	5.1683	0.0000	0.0080	0.1344	0.2296
ULS: 3. D + (S or Lr or R)	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0158	4.4761	-0.0000	0.0067	0.1135	0.1967
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 5b. D + 0.7E	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0158	4.4761	-0.0000	0.0067	0.1135	0.1967
ULS: 8. 0.6D + 0.7E	-0.0043	1.4397	-0.0000	0.0016	0.0304	0.0587
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6664	4.6288	-0.0055	-0.0131	0.0830	31.3110
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6521	0.1704	0.0054	0.0184	0.0180	-30.7898
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0103	6.1481	-0.0041	-0.0051	0.1377	23.6065
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0158	4.4761	-0.0000	0.0067	0.1135	0.1967
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9786	2.8042	0.0041	0.0185	0.0889	-22.9691
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0158	4.4761	-0.0000	0.0067	0.1135	0.1967
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0016	4.0715	-0.0041	-0.0092	0.0749	23.5077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9873	0.7277	0.0040	0.0145	0.0262	-23.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0071	2.3996	-0.0001	0.0026	0.0507	0.0978
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6636	3.6690	-0.0054	-0.0141	0.0627	31.2718
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0043	1.4397	-0.0000	0.0016	0.0304	0.0587
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6549	-0.7894	0.0054	0.0174	-0.0023	-30.8290
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0043	1.4397	-0.0000	0.0016	0.0304	0.0587

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	9.1672
Shear X	-4.4467
Shear Z	0.0091
Moment X	0.0321
Moment Y (Twist)	0.2215
Moment Z	52.5638

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	6.1481
Shear X	-2.6664
Shear Z	-0.0055
Moment X	0.0185
Moment Y (Twist)	0.1377
Moment Z	31.3110

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	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 2. D + L	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 3. D + (S or Lr or R)	0.0375	5.3900	-0.0000	0.0000	0.0000	-0.3674
ULS: 3. D + (S or Lr or R)	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0317	4.6630	-0.0000	0.0000	0.0000	-0.3075

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 5b. D + 0.7E	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0317	4.6630	-0.0000	0.0000	0.0000	-0.3075
ULS: 8. 0.6D + 0.7E	0.0085	1.4894	-0.0000	0.0000	0.0000	-0.0767
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6864	4.7527	-0.0000	0.0000	0.0000	31.5326
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.7151	0.2117	-0.0000	0.0000	0.0000	-31.4575
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9938	6.3659	-0.0000	0.0000	0.0000	23.4378
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0317	4.6630	-0.0000	0.0000	0.0000	-0.3075
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0573	2.9601	-0.0000	0.0000	0.0000	-23.8048
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0317	4.6630	-0.0000	0.0000	0.0000	-0.3075
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0113	4.1851	-0.0000	0.0000	0.0000	23.6175
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.0399	0.7794	-0.0000	0.0000	0.0000	-23.6251
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0142	2.4823	-0.0000	0.0000	0.0000	-0.1278
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6921	3.7598	-0.0000	0.0000	0.0000	31.5837
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0085	1.4894	-0.0000	0.0000	0.0000	-0.0767
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.7094	-0.7812	-0.0000	0.0000	0.0000	-31.4064
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0085	1.4894	-0.0000	0.0000	0.0000	-0.0767

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	9.5229
Shear X	-4.5297
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	52.8600

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	6.3659
Shear X	-2.7151
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	31.5837

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 2. D + L	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 3. D + (S or Lr or R)	-0.0188	5.1683	-0.0000	-0.0080	-0.1344	0.2296
ULS: 3. D + (S or Lr or R)	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0158	4.4761	0.0000	-0.0067	-0.1134	0.1967
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 5b. D + 0.7E	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0158	4.4761	0.0000	-0.0067	-0.1134	0.1967
ULS: 8. 0.6D + 0.7E	-0.0043	1.4397	0.0000	-0.0016	-0.0304	0.0587
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6664	4.6288	0.0055	0.0131	-0.0830	31.3110
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6521	0.1704	-0.0054	-0.0184	-0.0180	-30.7898
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978

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	-2.0103	6.1481	0.0041	0.0051	-0.1377	23.6066
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0158	4.4761	0.0000	-0.0067	-0.1134	0.1967
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9786	2.8042	-0.0041	-0.0185	-0.0889	-22.9690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0158	4.4761	0.0000	-0.0067	-0.1134	0.1967
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0016	4.0715	0.0041	0.0092	-0.0749	23.5077
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9873	0.7277	-0.0040	-0.0145	-0.0262	-23.0679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0071	2.3996	0.0001	-0.0026	-0.0507	0.0978
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6636	3.6690	0.0054	0.0142	-0.0627	31.2718
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0043	1.4397	0.0000	-0.0016	-0.0304	0.0587
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6549	-0.7894	-0.0054	-0.0174	0.0023	-30.8290
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0043	1.4397	0.0000	-0.0016	-0.0304	0.0587

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	9.1673
Shear X	-4.4468
Shear Z	-0.0091
Moment X	-0.0321
Moment Y (Twist)	0.2213
Moment Z	52.5644

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	6.1481
Shear X	-2.6664
Shear Z	0.0055
Moment X	-0.0185
Moment Y (Twist)	0.1377
Moment Z	31.3110

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Unit System: imperial

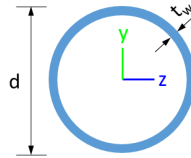


Design Input Information

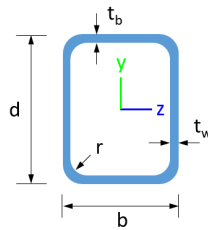
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Design Materials			
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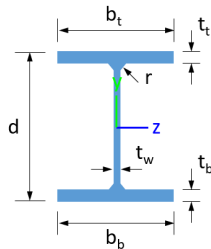
Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
11	10in Pipe Sch 40	10.75	0.36				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------

101	333.87	333.40	147.08	147.08	100.70	100.70
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	31.84	6.46	56.26	44.91
114	159.30	97.43	32.09	6.46	56.26	44.91
115	159.30	97.82	33.74	6.46	56.26	44.91
116	159.30	97.82	34.16	6.46	56.26	44.91
201	535.87	335.40	147.68	147.68	160.76	160.76
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	55.15	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	55.15	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	34.03	6.46	56.26	44.91
214	159.30	97.43	33.98	6.46	56.26	44.91
215	159.30	97.82	35.43	6.46	56.26	44.91
216	159.30	97.82	35.67	6.46	56.26	44.91

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	0.027	0.356	0.001	0.028	0.000	0.368	#13	0.400	Not Required	Pass
2	0.002	0.238	0.164	0.057	0.030	0.382	#13	0.036	Not Required	Pass
3	0.008	0.378	0.056	0.038	0.012	0.412	#13	0.046	Not Required	Pass
4	0.008	0.376	0.121	0.038	0.026	0.422	#13	0.082	Not Required	Pass
5	0.008	0.234	0.124	0.037	0.031	0.248	#13	0.076	Not Required	Pass
6	0.009	0.373	0.060	0.037	0.012	0.408	#13	0.046	Not Required	Pass
7	0.009	0.231	0.120	0.037	0.031	0.248	#13	0.076	Not Required	Pass
8	0.000	0.031	0.094	0.022	0.016	0.121	#21	0.102	Not Required	Pass
9	0.008	0.025	0.032	0.001	0.000	0.060	#13	0.206	Not Required	Pass
10	0.009	0.371	0.117	0.037	0.027	0.420	#13	0.082	Not Required	Pass
11	0.000	0.031	0.093	0.022	0.016	0.120	#21	0.102	Not Required	Pass
12	0.002	0.236	0.158	0.057	0.030	0.373	#13	0.036	Not Required	Pass
13	0.006	0.120	0.358	0.029	0.022	0.455	#21	0.306	Not Required	Pass
14	0.008	0.122	0.358	0.029	0.022	0.455	#21	0.204	Not Required	Pass
15	0.000	0.037	0.167	0.016	0.013	0.199	#21	Not Required	Not Required	Pass

16	0.000	0.037	0.167	0.016	0.013	0.199	#21	Not Required	Not Required	Pass
101	0.028	0.358	0.000	0.028	0.000	0.370	#13	0.400	Not Required	Pass
102	0.001	0.246	0.161	0.059	0.030	0.383	#13	0.036	Not Required	Pass
103	0.009	0.383	0.066	0.038	0.016	0.422	#13	0.046	Not Required	Pass
104	0.009	0.382	0.112	0.038	0.025	0.429	#13	0.082	Not Required	Pass
105	0.009	0.237	0.114	0.038	0.028	0.251	#13	0.076	Not Required	Pass
106	0.009	0.383	0.066	0.038	0.016	0.422	#13	0.046	Not Required	Pass
107	0.009	0.237	0.114	0.038	0.028	0.251	#13	0.076	Not Required	Pass
108	0.000	0.037	0.095	0.020	0.016	0.127	#21	0.102	Not Required	Pass
109	0.006	0.022	0.031	0.001	0.000	0.055	#13	0.206	Not Required	Pass
110	0.009	0.382	0.112	0.038	0.025	0.429	#13	0.082	Not Required	Pass
111	0.000	0.037	0.094	0.020	0.016	0.125	#21	0.102	Not Required	Pass
112	0.001	0.246	0.161	0.059	0.030	0.383	#13	0.036	Not Required	Pass
113	0.005	0.086	0.321	0.027	0.022	0.388	#21	0.306	Not Required	Pass
114	0.008	0.088	0.320	0.027	0.022	0.385	#21	0.306	Not Required	Pass
115	0.000	0.072	0.179	0.020	0.016	0.242	#21	0.370	Not Required	Pass
116	0.000	0.072	0.180	0.020	0.016	0.244	#21	0.370	Not Required	Pass
201	0.027	0.356	0.001	0.028	0.000	0.368	#13	0.400	Not Required	Pass
202	0.002	0.236	0.158	0.057	0.030	0.373	#13	0.036	Not Required	Pass
203	0.009	0.373	0.060	0.037	0.012	0.408	#13	0.046	Not Required	Pass
204	0.009	0.371	0.117	0.037	0.027	0.420	#13	0.082	Not Required	Pass
205	0.009	0.231	0.120	0.037	0.031	0.248	#13	0.076	Not Required	Pass
206	0.008	0.378	0.056	0.038	0.012	0.412	#13	0.046	Not Required	Pass
207	0.008	0.234	0.124	0.037	0.031	0.248	#13	0.076	Not Required	Pass
208	0.000	0.037	0.167	0.016	0.013	0.199	#21	Not Required	Not Required	Pass
209	0.008	0.025	0.032	0.001	0.000	0.060	#13	0.206	Not Required	Pass
210	0.008	0.376	0.121	0.038	0.026	0.422	#13	0.082	Not Required	Pass
211	0.000	0.037	0.167	0.016	0.013	0.199	#21	Not Required	Not Required	Pass
212	0.002	0.238	0.164	0.057	0.030	0.382	#13	0.036	Not Required	Pass
213	0.006	0.120	0.358	0.029	0.022	0.455	#21	0.204	Not Required	Pass
214	0.008	0.122	0.358	0.029	0.022	0.455	#21	0.306	Not Required	Pass
215	0.000	0.070	0.179	0.022	0.016	0.240	#21	0.370	Not Required	Pass
216	0.000	0.070	0.180	0.022	0.016	0.242	#21	0.370	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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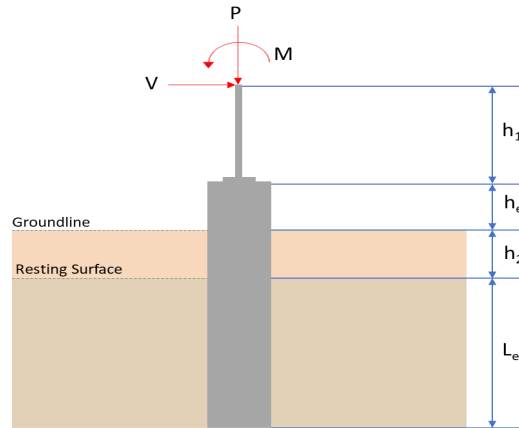
SkyCiv Foundation Design

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

$L = 6.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.148	9.167
V_x (kip)	-2.666	-4.447
V_z (kip)	-0.005	0.009
M_x (kipft)	0.019	0.032
M_z (kipft)	31.311	52.564

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,

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

$$H_o = \frac{(-2.666 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42452 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.311 \text{ kipft}) + ((-2.666 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9858 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.2185 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.019 \text{ kipft}) + ((-0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0030255 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.59787 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.2185 \text{ ft}), (0.59787 \text{ ft})]$$

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

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

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

$$\text{Ratio} = 0.92133$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.148 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38425 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38425 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19212$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42452 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.9858 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9858 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9858 \text{ kipft/ft})) + (4 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6558 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9858 \text{ kipft/ft})) + (3 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9858 \text{ kipft/ft})) + (2 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.2297 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.9858 \text{ kipft/ft})) + ((-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.93579 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6558 \text{ ft})}{2}$$

$$p_a = 0.34919 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2297 \text{ kip/ft}^2)}{(0.34919 \text{ kip/ft}^2)}$$

$$Ratio = 0.65782$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.93579 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.92423$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = -0.00079618 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0030255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0030255 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0030255 \text{ kipft/ft})) + (4 \times (-0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

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

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

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

$$p = \frac{0.75 \times [(4 \times (0.0030255 \text{ kipft/ft})) + (3 \times (-0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.0030255 \text{ kipft/ft})) + (2 \times (-0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = -0.00015916 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0030255 \text{ kipft/ft})) + ((-0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.00008912 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

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

$$p_a = 0.36037 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00015916 \text{ kip/ft}^2)}{(0.36037 \text{ kip/ft}^2)}$$

$$Ratio = -0.00044165$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

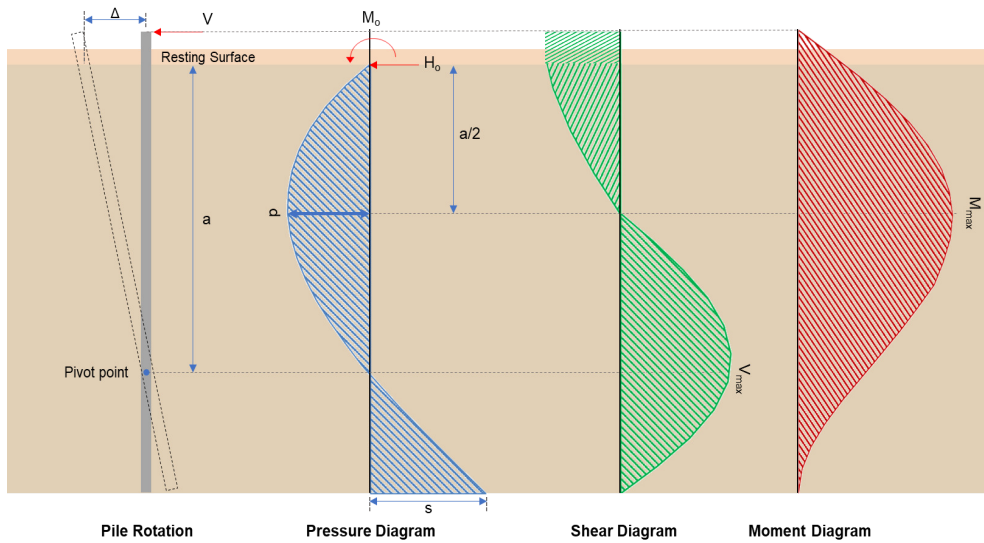
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00008912 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.000088019$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.447 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70812 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.564 \text{ kipft}) + ((-4.447 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3701 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.3701 \text{ kipft/ft})}{(-0.70812 \text{ kip/ft})}$$

$$E = 11.82 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3701 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{6 \times (8.3701 \text{ kipft/ft}) + 4 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft})}$$

$$a = \frac{(-0.70812 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (8.3701 \text{ kipft/ft})) + (4 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6551 \text{ ft}$$

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

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

$$V_{max} = ((-0.70812 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.645 \text{ kip}$$

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

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

$$M_{max} = ((-0.70812 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.82 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.191 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.009 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0014331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.032 \text{ kipft}) + ((0.009 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0050955 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0050955 \text{ kipft/ft})}{(0.0014331 \text{ kip/ft})}$$

$$E = 3.5556 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0050955 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0050955 \text{ kipft/ft})) + (4 \times (0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8142 \text{ ft}$$

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

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

$$V_{max} = ((0.0014331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8142 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8142 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0091595 \text{ kip}$$

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

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

$$M_{max} = ((0.0014331 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.5556 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.02746 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.167 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.292 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.292 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</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>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</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;">$Ratio = \frac{(9.167 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0034267$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.167 \text{ kip} \rightarrow 9167 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9167 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

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

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

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

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

Considering x-direction:

V_{max} = 10.645 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.645 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.09597$$

Considering z-direction:

$V_{max} = 0.0091595 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.0091595 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.000082599$$

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

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:

$M_{max} = 34.191 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(34.191 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13698$$

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

Considering z-direction:

$M_{max} = 0.02746 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.02746 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00011001$$

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

REFERENCES	CALCULATIONS	RESULTS
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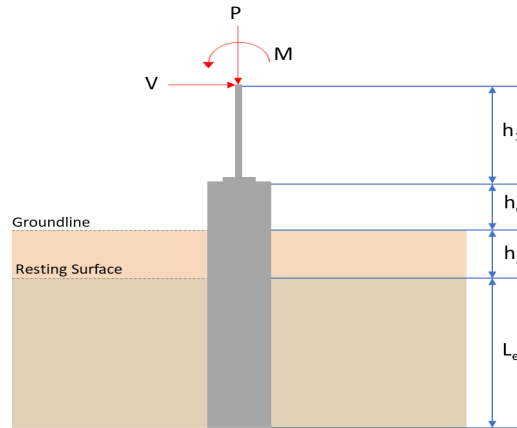
SkyCiv Foundation Design

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

$L = 6.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.366	9.523
V_x (kip)	-2.715	-4.530
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	31.584	52.860

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,

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

$$H_o = \frac{(-2.715 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.43232 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.584 \text{ kipft}) + ((-2.715 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0293 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.2225 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(6.2225 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.223 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.223 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.92193$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.366 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39787 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39787 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19894$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.43232 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.0293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0293 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.43232 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.0293 \text{ kipft/ft})) + (4 \times (-0.43232 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6569 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.0293 \text{ kipft/ft})) + (3 \times (-0.43232 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (5.0293 \text{ kipft/ft})) + (2 \times (-0.43232 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.22972 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.0293 \text{ kipft/ft})) + ((-0.43232 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.9403 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6569 \text{ ft})}{2}$$

$$p_a = 0.34927 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22972 \text{ kip/ft}^2)}{(0.34927 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65772$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

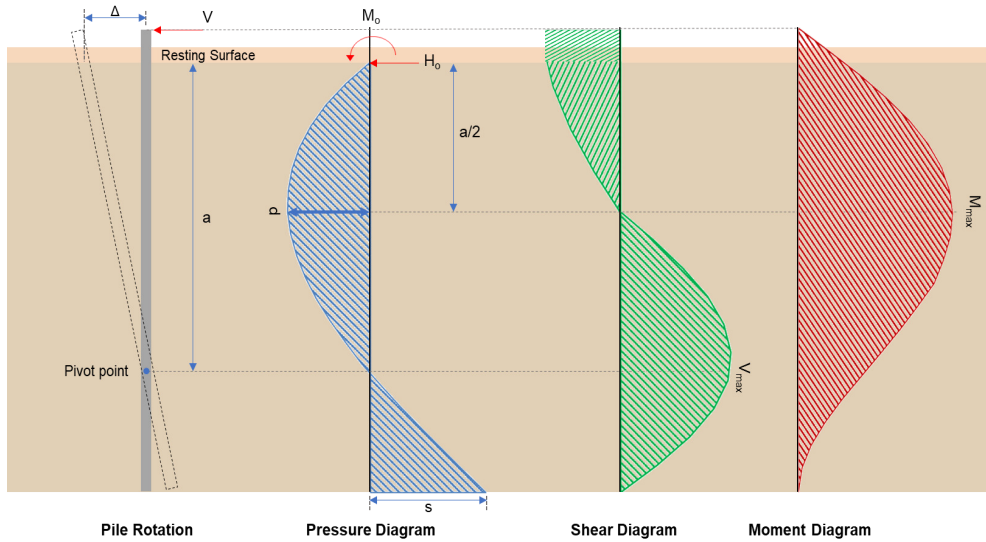
$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.9403 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.660**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.53 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.72134 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.86 \text{ kipft}) + ((-4.53 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.4172 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.4172 \text{ kipft/ft})}{(-0.72134 \text{ kip/ft})}$$

$$E = 11.669 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.4172 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.72134 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.4172 \text{ kipft/ft})) + (4 \times (-0.72134 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6566 \text{ ft}$$

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

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

$$V_{max} = ((-0.72134 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.669 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6566 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.669 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6566 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.129 \text{ kip}$$

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

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

$$M_{max} = ((-0.72134 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.669 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6566 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.669 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6566 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (11.669 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6566 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.443 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{9.523 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.28 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.28 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(9.523 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0035598$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 $b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.523 \text{ kip} \rightarrow 9523 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9523 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.76 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.76 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.76 \text{ kip}$$

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)

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

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

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

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

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

$$\phi V_n = (0.65) \times ((119.76 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.92 \text{ kip}$$

Considering x-direction:

$V_{max} = 10.729 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(10.729 \text{ kip})}{(110.92 \text{ kip})}$$

$$\text{Ratio} = 0.096729$$

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

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

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

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

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

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

Allowable flexural strength:

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

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

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

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

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: $M_{max} = 34.443 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(34.443 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.13799$$

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

REFERENCES	CALCULATIONS	RESULTS
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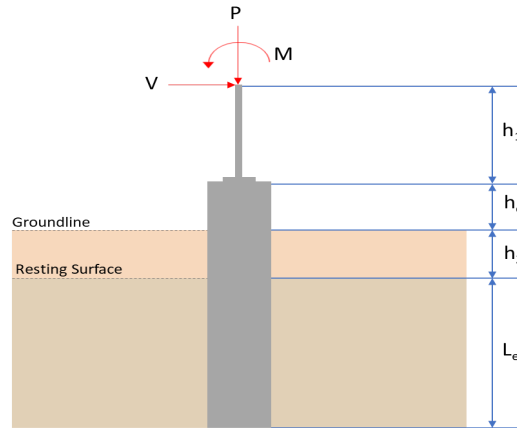
SkyCiv Foundation Design

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

$L = 6.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.148	9.167
V_x (kip)	-2.666	-4.447
V_z (kip)	0.005	-0.009
M_x (kipft)	-0.019	-0.032
M_z (kipft)	31.311	52.564

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,

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

$$H_o = \frac{(-2.666 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42452 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.311 \text{ kipft}) + ((-2.666 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9858 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.2185 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.005 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.00079618 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.019 \text{ kipft}) + ((0.005 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0030255 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.64916 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.2185 \text{ ft}), (0.64916 \text{ ft})]$$

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

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

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

$$\text{Ratio} = 0.92133$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.148 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38425 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38425 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19212$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42452 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.9858 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9858 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.9858 \text{ kipft/ft})) + (4 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6558 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.9858 \text{ kipft/ft})) + (3 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.9858 \text{ kipft/ft})) + (2 \times (-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.2297 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.9858 \text{ kipft/ft})) + ((-0.42452 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.93579 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6558 \text{ ft})}{2}$$

$$p_a = 0.34919 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.2297 \text{ kip/ft}^2)}{(0.34919 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.65782$$

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

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

$$\text{Ratio} = \frac{(0.93579 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92423$$

Status: **PASS**
Ratio: **0.660**

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

Considering z-direction:

$H_o = 0.00079618 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0030255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0030255 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0030255 \text{ kipft/ft})) + (4 \times (0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

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

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

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

$$p = \frac{0.75 \times [(4 \times (0.0030255 \text{ kipft/ft})) + (3 \times (0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.0030255 \text{ kipft/ft})) + (2 \times (0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.00066145 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0030255 \text{ kipft/ft})) + ((0.00079618 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.0015045 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

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

$$p_a = 0.36037 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00066145 \text{ kip/ft}^2)}{(0.36037 \text{ kip/ft}^2)}$$

$$Ratio = 0.0018355$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

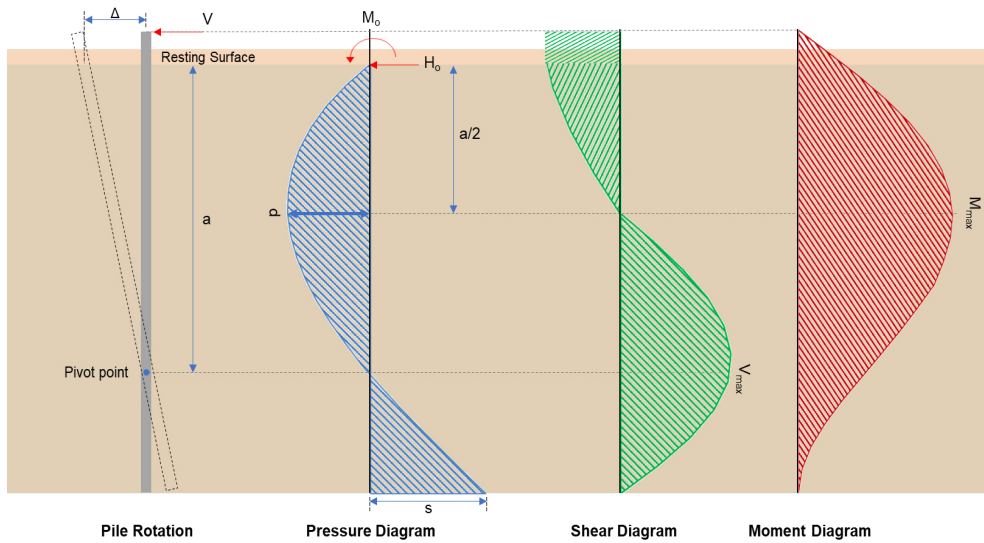
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0015045 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.001486$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.447 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70812 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.564 \text{ kipft}) + ((-4.447 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.3701 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.3701 \text{ kipft/ft})}{(-0.70812 \text{ kip/ft})}$$

$$E = 11.82 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.3701 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (8.3701 \text{ kipft/ft})) + (4 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(-0.70812 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (8.3701 \text{ kipft/ft})) + (4 \times (-0.70812 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6551 \text{ ft}$$

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

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

$$V_{max} = ((-0.70812 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.645 \text{ kip}$$

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

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

$$M_{max} = ((-0.70812 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(11.82 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.82 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6551 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.191 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.009 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0014331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.032 \text{ kipft}) + ((-0.009 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0050955 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0050955 \text{ kipft/ft})}{(-0.0014331 \text{ kip/ft})}$$

$$E = 3.5556 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0050955 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0050955 \text{ kipft/ft})) + (4 \times (-0.0014331 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8142 \text{ ft}$$

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

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

$$V_{max} = ((-0.0014331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8142 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8142 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0091595 \text{ kip}$$

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

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

$$M_{max} = ((-0.0014331 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(3.5556 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.5556 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.8142 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.02746 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.167 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.292 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.292 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio - Capacity</i></p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.167 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0034267$	<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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.167 \text{ kip} \rightarrow 9167 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9167 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

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

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

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

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

Considering x-direction:

V_{max} = 10.645 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.645 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.095997$$

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

Considering z-direction:

$V_{max} = 0.0091595 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.0091595 \text{ kip})}{(110.89 \text{ kip})}$$

$$Ratio = 0.000082599$$

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:

$M_{max} = 34.191 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.191 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.13698$$

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

Considering z-direction:

$M_{max} = 0.02746 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.02746 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00011001$$

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