

Your Project Calculations



Project Name: Test-RevA

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Test-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=dW4IHmupg70cFp3nspz5Kcijgg7g0PiyWRkKzSMXdzV5OWEbY0s77LIGyWTywt

Array Specification

Product:	Beam
Unique ID:	3P-22.5-6TOP-SD-72-L-5Hx10W-C8DB
Duty Classification:	SD
Module Width:	42.00 in
Module Length:	78.00in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	10
Front Edge Clearance:	9
Total Array Height at Tilt:	12.06 ft
Total Frame Length:	64.50 ft
Frame Weight:	2221 lbs
Array Dimensions N/S:	17.71 ft
Array Dimensions E/W:	65.83 ft
Rail Length:	212.50 in
Rail Spacing:	3.25 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.54 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.00 ft Pile 2: 5.25 ft Pile 3: 5.00 ft
Foundation Volume:	9.037 y ³
Foundation Result:	PASSED
Mount Twist:	0.035192 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	117 Pinon Dr, Portola Valley, CA 94028, USA
Wind Speed:	86 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

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

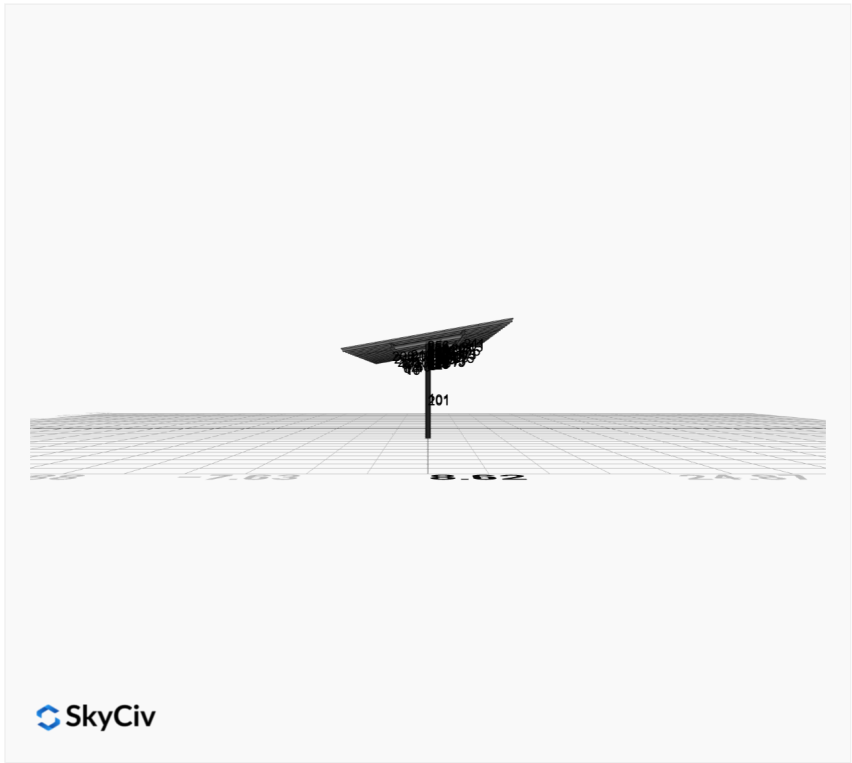
AutoDesigner Input

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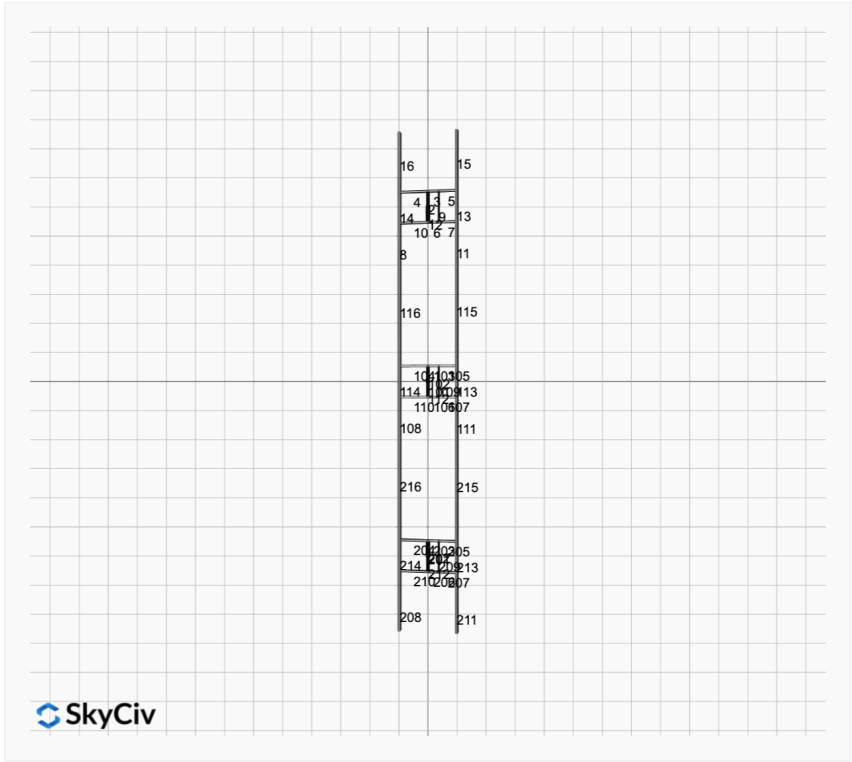
Design Notes:


- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

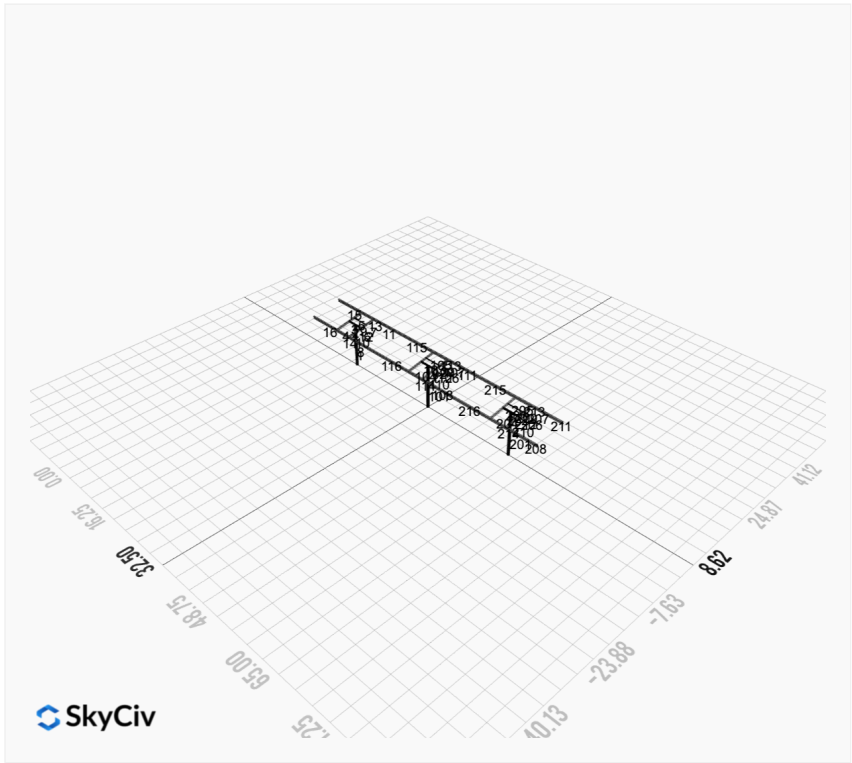




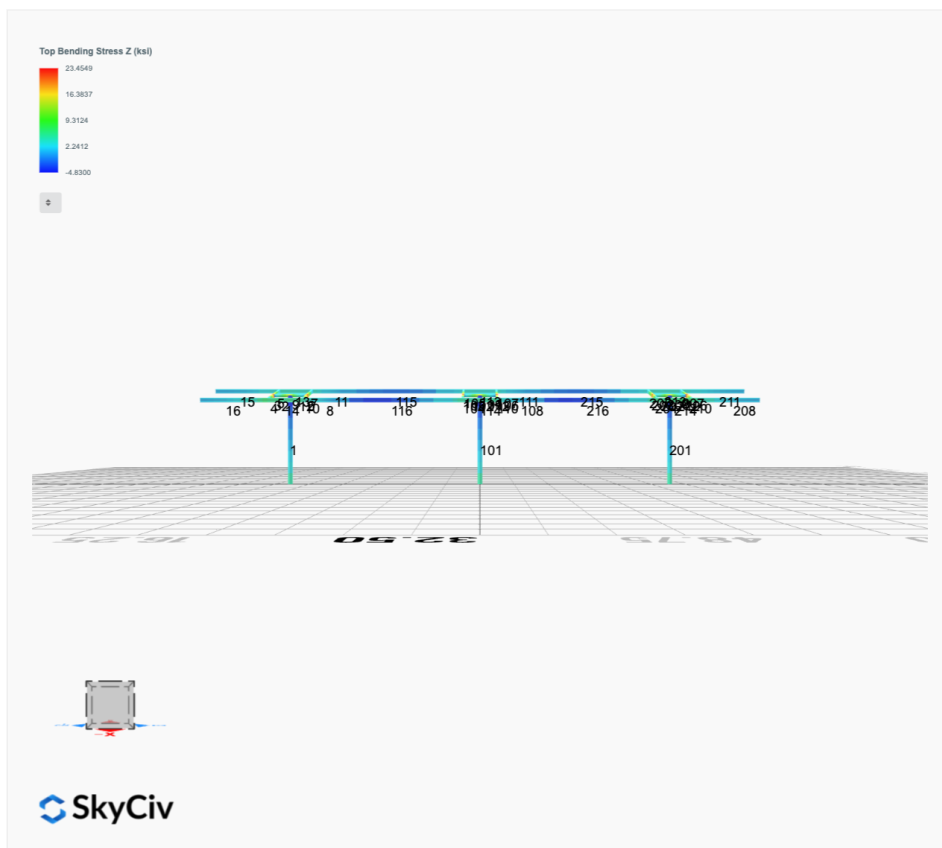
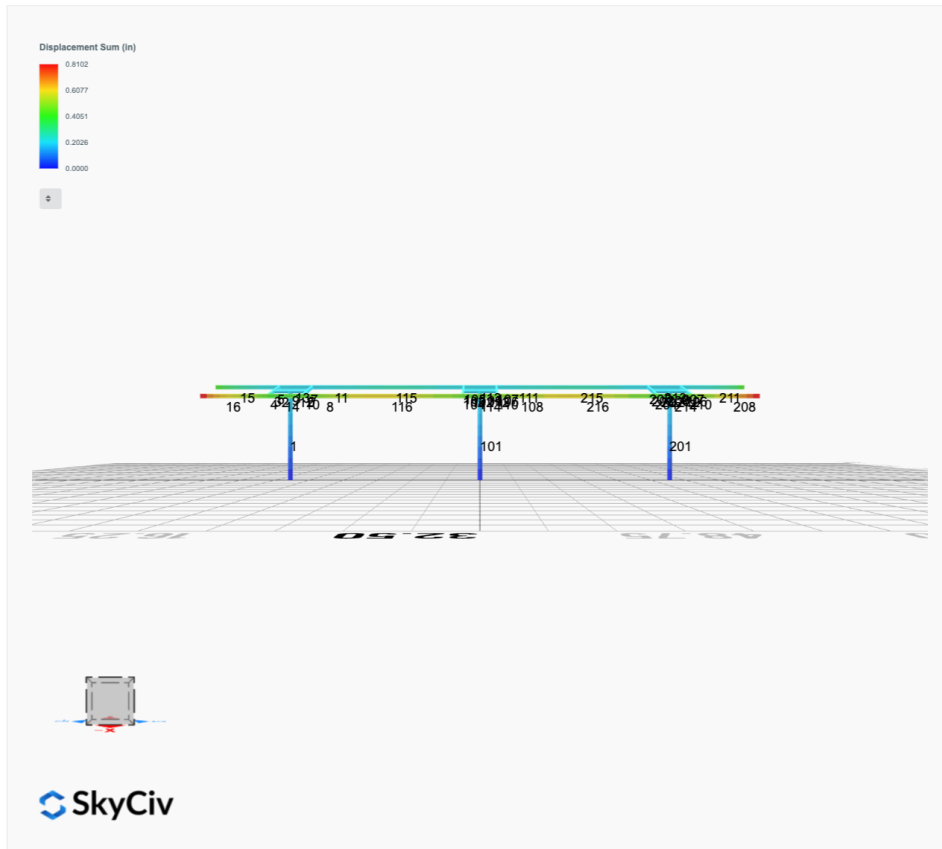
 SkyCiv

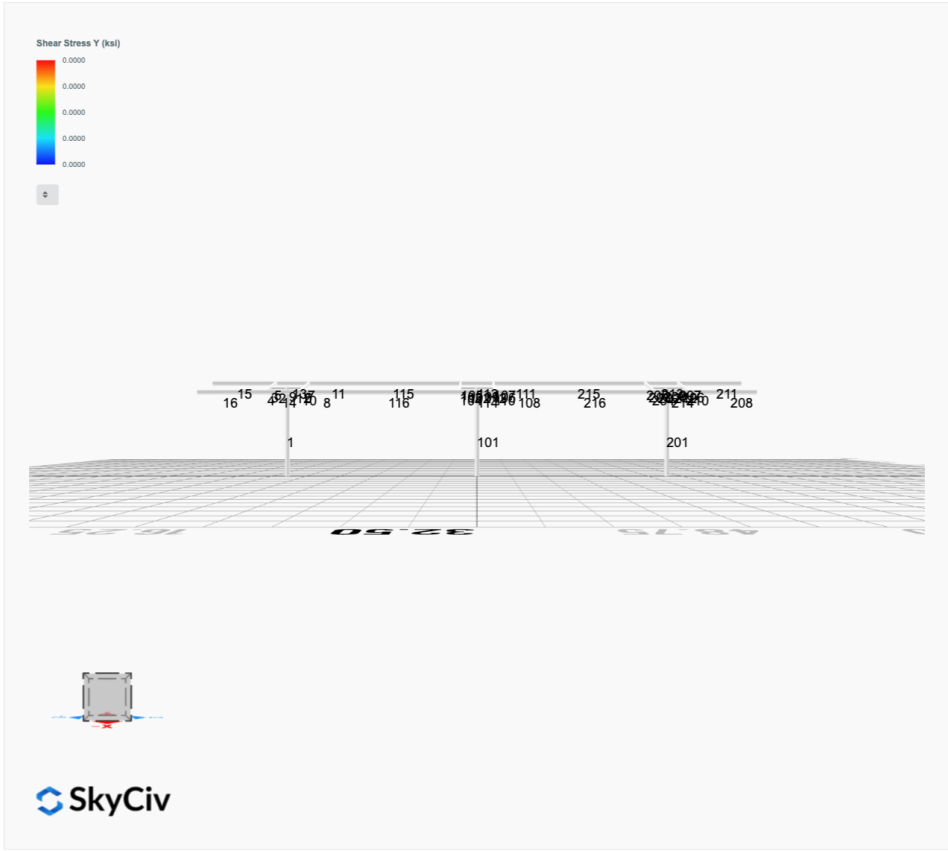
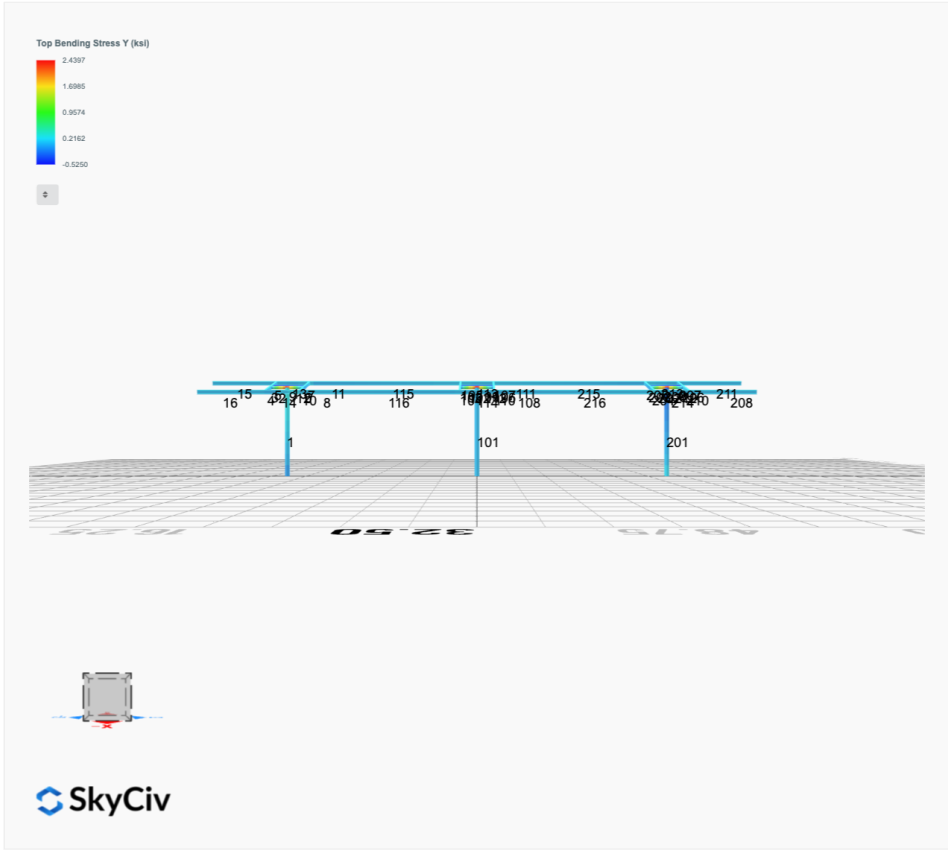


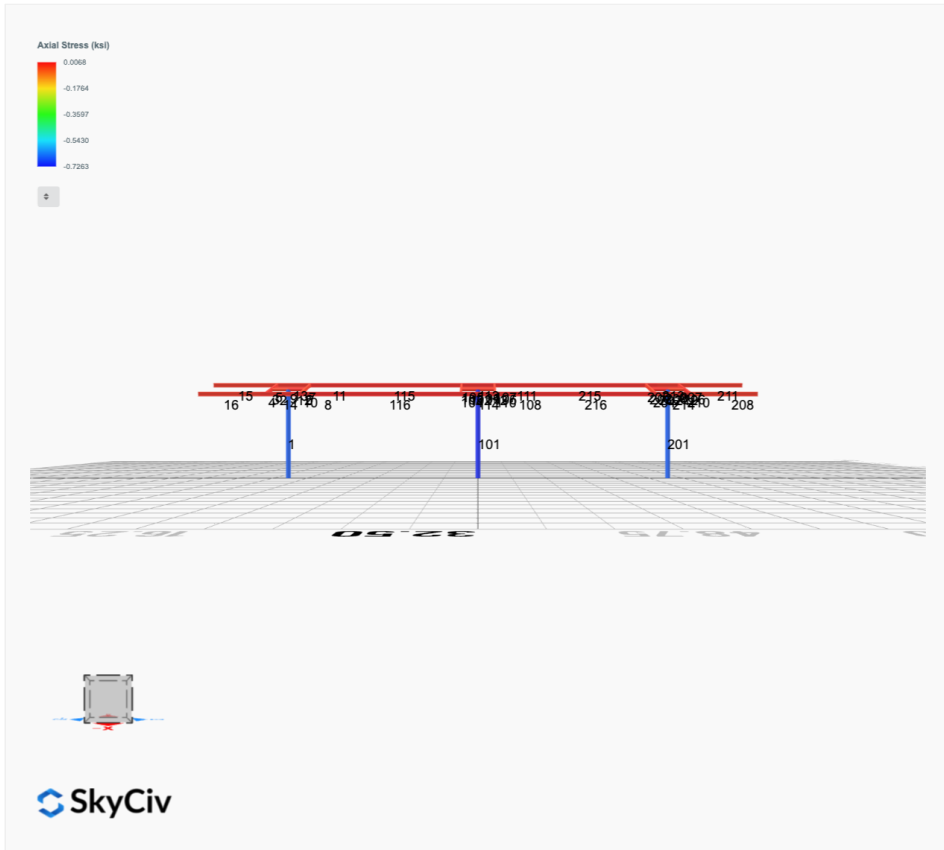
 SkyCiv



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	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 2. D + L	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 3. D + (S or Lr or R)	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 3. D + (S or Lr or R)	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 5b. D + 0.7E	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0023	2.4606	-0.0177	-0.0566	0.0126	0.0441
ULS: 8. 0.6D + 0.7E	-0.0014	1.4764	-0.0106	-0.0339	0.0076	0.0265
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5893	5.7715	-0.0448	-0.1437	0.0247	9.0745
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5893	5.7715	-0.0448	-0.1437	0.0247	9.0745
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4076	0.1391	0.0019	0.0062	0.0026	-2.2753
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3582	0.4392	-0.0023	-0.0067	0.0072	-11.1496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4426	4.9438	-0.0380	-0.1219	0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4426	4.9438	-0.0380	-0.1219	0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3051	0.7195	-0.0030	-0.0095	0.0051	-1.6954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2680	0.9446	-0.0061	-0.0192	0.0085	-8.3512
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4426	4.9438	-0.0380	-0.1219	0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4426	4.9438	-0.0380	-0.1219	0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3051	0.7195	-0.0030	-0.0095	0.0051	-1.6954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2680	0.9446	-0.0061	-0.0192	0.0085	-8.3512
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5884	4.7873	-0.0377	-0.1211	0.0197	9.0569
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5884	4.7873	-0.0377	-0.1211	0.0197	9.0569
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4085	-0.8451	0.0090	0.0288	-0.0024	-2.2929
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3591	-0.5450	0.0048	0.0159	0.0021	-11.1672

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	8.4710
Shear X	-0.9813
Shear Z	-0.0664
Moment X	-0.2139
Moment Y (Twist)	0.0352
Moment Z	19.0915

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	5.7715
Shear X	-0.5893
Shear Z	-0.0448
Moment X	-0.1437
Moment Y (Twist)	0.0247
Moment Z	11.1672

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	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 2. D + L	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 3. D + (S or Lr or R)	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 3. D + (S or Lr or R)	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 5b. D + 0.7E	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0047	2.5633	-0.0000	0.0000	-0.0000	-0.0204
ULS: 8. 0.6D + 0.7E	0.0028	1.5380	-0.0000	0.0000	-0.0000	-0.0122
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6003	6.0300	-0.0000	0.0000	-0.0000	9.2961
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6003	6.0300	-0.0000	0.0000	-0.0000	9.2961
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4324	0.1312	-0.0000	0.0000	-0.0000	-2.4366
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3695	0.4481	-0.0000	0.0000	-0.0000	-11.5098
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4490	5.1633	-0.0000	0.0000	-0.0000	6.9670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4490	5.1633	-0.0000	0.0000	-0.0000	6.9670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3255	0.7392	-0.0000	0.0000	-0.0000	-1.8325
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2783	0.9769	-0.0000	0.0000	-0.0000	-8.6374
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4490	5.1633	-0.0000	0.0000	-0.0000	6.9670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4490	5.1633	-0.0000	0.0000	-0.0000	6.9670
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3255	0.7392	-0.0000	0.0000	-0.0000	-1.8325
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2783	0.9769	-0.0000	0.0000	-0.0000	-8.6374
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6021	5.0047	-0.0000	0.0000	-0.0000	9.3043
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6021	5.0047	-0.0000	0.0000	-0.0000	9.3043
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4305	-0.8942	-0.0000	0.0000	-0.0000	-2.4284
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3676	-0.5772	-0.0000	0.0000	-0.0000	-11.5016

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	8.8537
Shear X	-1.0082
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	19.6832

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	6.0300
Shear X	-0.6021
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	11.5098

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	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 2. D + L	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 3. D + (S or Lr or R)	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 3. D + (S or Lr or R)	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 5b. D + 0.7E	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0023	2.4606	0.0177	0.0566	-0.0126	0.0441
ULS: 8. 0.6D + 0.7E	-0.0014	1.4764	0.0106	0.0339	-0.0076	0.0264
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5893	5.7715	0.0448	0.1437	-0.0247	9.0745
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5893	5.7715	0.0448	0.1437	-0.0247	9.0745
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4076	0.1391	-0.0019	-0.0062	-0.0026	-2.2753
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3582	0.4392	0.0023	0.0067	-0.0072	-11.1496
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4426	4.9438	0.0380	0.1219	-0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4426	4.9438	0.0380	0.1219	-0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3051	0.7195	0.0030	0.0095	-0.0051	-1.6954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2680	0.9446	0.0061	0.0192	-0.0085	-8.3512

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	-0.4426	4.9438	0.0380	0.1219	-0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4426	4.9438	0.0380	0.1219	-0.0217	6.8169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3051	0.7195	0.0030	0.0095	-0.0051	-1.6954
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2680	0.9446	0.0061	0.0192	-0.0085	-8.3512
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5884	4.7873	0.0377	0.1211	-0.0197	9.0569
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5884	4.7873	0.0377	0.1211	-0.0197	9.0569
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4085	-0.8451	-0.0090	-0.0288	0.0024	-2.2929
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3591	-0.5450	-0.0048	-0.0159	-0.0021	-11.1672

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	8.4710
Shear X	-0.9813
Shear Z	0.0664
Moment X	0.2139
Moment Y (Twist)	0.0352
Moment Z	19.0917

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	5.7715
Shear X	-0.5893
Shear Z	0.0448
Moment X	0.1437
Moment Y (Twist)	0.0247
Moment Z	11.1672

Project Details

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



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions



ID	Name	d (in)	t_w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	90.16	42.30	42.30	75.35	75.35
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.01	6.45	30.09	45.74
14	120.60	98.23	19.01	6.45	30.09	45.74
15	120.60	21.74	23.36	6.45	30.09	45.74
16	120.60	21.74	23.36	6.45	30.09	45.74
101	251.16	90.16	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.13	6.45	30.09	45.74
114	120.60	98.23	17.96	6.45	30.09	45.74
115	120.60	48.60	11.32	6.45	30.09	45.74
116	120.60	48.60	11.22	6.45	30.09	45.74
201	251.16	90.16	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61
208	120.60	21.74	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	21.74	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	19.01	6.45	30.09	45.74
214	120.60	98.23	19.01	6.45	30.09	45.74
215	120.60	48.60	11.72	6.45	30.09	45.74
216	120.60	48.60	11.52	6.45	30.09	45.74

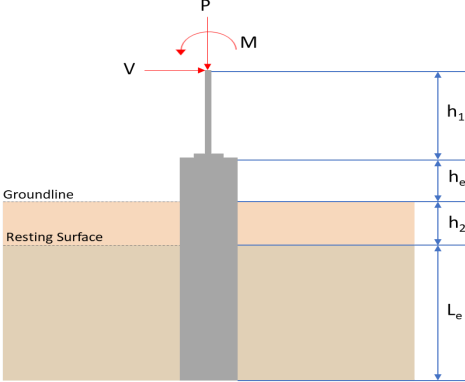
Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.094	0.451	0.011	0.013	0.001	0.453	#16	0.591	Not Required	Pass
2	0.001	0.484	0.067	0.098	0.012	0.551	#13	0.034	Not Required	Pass
3	0.002	0.805	0.018	0.082	0.001	0.824	#13	0.044	Not Required	Pass
4	0.002	0.617	0.052	0.062	0.006	0.637	#13	0.078	Not Required	Pass
5	0.002	0.499	0.058	0.081	0.009	0.514	#13	0.073	Not Required	Pass
6	0.002	0.778	0.013	0.078	0.001	0.785	#13	0.044	Not Required	Pass
7	0.002	0.483	0.055	0.078	0.008	0.492	#13	0.073	Not Required	Pass
8	0.000	0.073	0.024	0.042	0.002	0.091	#13	0.088	Not Required	Pass
9	0.003	0.097	0.021	0.001	0.000	0.120	#13	0.198	Not Required	Pass
10	0.002	0.588	0.056	0.060	0.007	0.626	#13	0.078	Not Required	Pass
11	0.000	0.099	0.025	0.055	0.002	0.116	#13	0.088	Not Required	Pass
12	0.001	0.454	0.064	0.094	0.011	0.519	#13	0.034	Not Required	Pass
13	0.001	0.344	0.057	0.068	0.002	0.391	#13	0.265	Not Required	Pass
14	0.001	0.273	0.057	0.052	0.002	0.310	#13	0.177	Not Required	Pass
15	0.000	0.167	0.034	0.043	0.002	0.196	#13	Not Required	Not Required	Pass
16	0.000	0.128	0.034	0.033	0.002	0.157	#13	Not Required	Not Required	Pass
101	0.098	0.465	0.000	0.013	0.000	0.466	#16	0.591	Not Required	Pass
102	0.001	0.484	0.066	0.101	0.012	0.550	#13	0.034	Not Required	Pass
103	0.002	0.827	0.017	0.084	0.001	0.842	#13	0.044	Not Required	Pass
104	0.002	0.632	0.050	0.064	0.006	0.663	#13	0.078	Not Required	Pass
105	0.002	0.513	0.053	0.083	0.008	0.526	#13	0.073	Not Required	Pass
106	0.002	0.827	0.017	0.084	0.001	0.842	#13	0.044	Not Required	Pass
107	0.002	0.513	0.053	0.083	0.008	0.526	#13	0.073	Not Required	Pass
108	0.000	0.057	0.023	0.040	0.002	0.080	#13	0.088	Not Required	Pass
109	0.003	0.086	0.017	0.001	0.000	0.104	#13	0.198	Not Required	Pass
110	0.002	0.632	0.050	0.064	0.006	0.663	#13	0.078	Not Required	Pass
111	0.000	0.073	0.023	0.053	0.002	0.097	#13	0.088	Not Required	Pass
112	0.001	0.484	0.066	0.101	0.012	0.550	#13	0.034	Not Required	Pass
113	0.001	0.275	0.051	0.065	0.002	0.314	#13	0.265	Not Required	Pass
114	0.001	0.223	0.051	0.050	0.002	0.252	#13	0.265	Not Required	Pass
115	0.000	0.354	0.029	0.053	0.002	0.379	#13	0.557	Not Required	Pass
116	0.000	0.270	0.029	0.040	0.002	0.296	#13	0.557	Not Required	Pass
201	0.094	0.451	0.011	0.013	0.001	0.453	#16	0.591	Not Required	Pass
202	0.001	0.454	0.064	0.094	0.011	0.519	#13	0.034	Not Required	Pass
203	0.002	0.778	0.013	0.078	0.001	0.785	#13	0.044	Not Required	Pass
204	0.002	0.588	0.056	0.060	0.007	0.626	#13	0.078	Not Required	Pass
205	0.002	0.483	0.055	0.078	0.008	0.492	#13	0.073	Not Required	Pass
206	0.002	0.805	0.018	0.082	0.001	0.824	#13	0.044	Not Required	Pass
207	0.002	0.499	0.058	0.081	0.009	0.514	#13	0.073	Not Required	Pass
208	0.000	0.128	0.034	0.033	0.002	0.157	#13	Not Required	Not Required	Pass
209	0.003	0.097	0.021	0.001	0.000	0.120	#13	0.198	Not Required	Pass
210	0.002	0.617	0.052	0.062	0.006	0.637	#13	0.078	Not Required	Pass
211	0.000	0.167	0.034	0.043	0.002	0.196	#13	Not Required	Not Required	Pass
212	0.001	0.484	0.067	0.098	0.012	0.551	#13	0.034	Not Required	Pass
213	0.001	0.344	0.057	0.068	0.002	0.391	#13	0.177	Not Required	Pass
214	0.001	0.273	0.057	0.052	0.002	0.310	#13	0.265	Not Required	Pass
215	0.000	0.349	0.029	0.055	0.002	0.374	#13	0.557	Not Required	Pass
216	0.000	0.266	0.029	0.042	0.002	0.291	#13	0.557	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																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.772</td> <td>8.471</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.589</td> <td>-0.981</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.045</td> <td>-0.066</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.144</td> <td>-0.214</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.167</td> <td>19.092</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_n) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.772	8.471	V_x (kip)	-0.589	-0.981	V_z (kip)	-0.045	-0.066	M_x (kipft)	-0.144	-0.214	M_z (kipft)	11.167	19.092	
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M_z (kipft)	11.167	19.092																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.589 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.09379 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.167 \text{ kipft}) + ((-0.589 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7782 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8615 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.045 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0071656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.144 \text{ kipft}) + ((-0.045 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.02293 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1074 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8615 \text{ ft}), (1.1074 \text{ ft})]$$

$$L_{e,req} = 4.861 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.861 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9722$$

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

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

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

$$q = \frac{(5.772 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36075 \text{ kip/ft}^2$$

$$q = 0.00010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36075 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18038$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.09379 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7782 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7782 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.7782 \text{ kipft/ft})) + (4 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3956 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7782 \text{ kipft/ft})) + (3 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.7782 \text{ kipft/ft})) + (2 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.22215 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.7782 \text{ kipft/ft})) + ((-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74098 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3956 \text{ ft})}{2}$$

$$p_a = 0.25467 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22215 \text{ kip/ft}^2)}{(0.25467 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8723$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.870**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.74098 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98797$$

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

Considering z-direction:

$H_o = -0.0071656 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.02293 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.02293 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.0071656 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.02293 \text{ kipft/ft})) + (4 \times (-0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5459 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.02293 \text{ kipft/ft})) + (3 \times (-0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.02293 \text{ kipft/ft})) + (2 \times (-0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = -0.0026011 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.02293 \text{ kipft/ft})) + ((-0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.0024076 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5459 \text{ ft})}{2}$$

$$p_a = 0.26594 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0026011 \text{ kip/ft}^2)}{(0.26594 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0097807$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

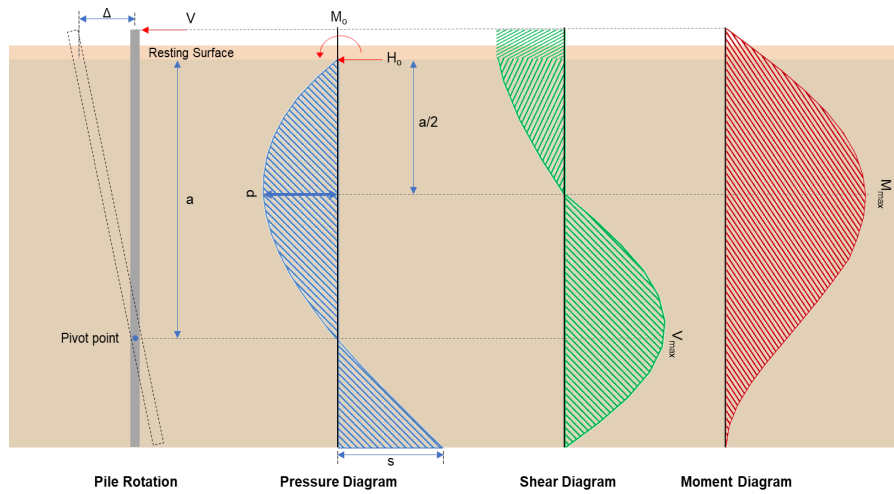
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0024076 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.0032102$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.981 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15621 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.092 \text{ kipft}) + ((-0.981 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0401 \text{ kipft/ft})}{(-0.15621 \text{ kip/ft})}$$

$$E = 19.462 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0401 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.15621 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.0401 \text{ kipft/ft})) + (4 \times (-0.15621 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3943 \text{ ft}$$

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

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

$$V_{max} = ((-0.15621 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3943 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3943 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7223 \text{ kip}$$

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

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

$$M_{max} = ((-0.15621 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(19.462 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.519 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.066 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01051 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.214 \text{ kipft}) + ((-0.066 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.034076 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.034076 \text{ kipft/ft})}{(-0.01051 \text{ kip/ft})}$$

$$E = 3.2424 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034076 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.01051 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.034076 \text{ kipft/ft})) + (4 \times (-0.01051 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5445 \text{ ft}$$

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

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

$$V_{max} = ((-0.01051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5445 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5445 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.076142 \text{ kip}$$

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

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

$$M_{max} = ((-0.01051 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(3.2424 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.17154 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.471 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.315 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.315 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

$$\text{Ratio} = \frac{(8.471 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0031665$$

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

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.471 \text{ kip} \rightarrow 8471 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8471 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.61 \text{ kip}$$

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

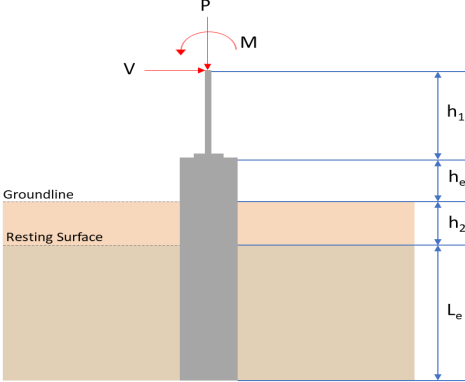
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.61 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.61 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.61 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.83 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.7223 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.7223 \text{ kip})}{(110.83 \text{ kip})}$ $\text{Ratio} = 0.042608$ <p>Considering z-direction:</p> <p>$V_{max} = 0.076142 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.076142 \text{ kip})}{(110.83 \text{ kip})}$ $\text{Ratio} = 0.00068701$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 11.519 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.519 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.046152$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.17154 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.17154 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00068725$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.772</td> <td>8.471</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.589</td> <td>-0.981</td> </tr> <tr> <td>V_z (kip)</td> <td>0.045</td> <td>0.066</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.144</td> <td>0.214</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.167</td> <td>19.092</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	5.772	8.471	V_x (kip)	-0.589	-0.981	V_z (kip)	0.045	0.066	M_x (kipft)	0.144	0.214	M_z (kipft)	11.167	19.092	
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M_x (kipft)	0.144	0.214																										
M_z (kipft)	11.167	19.092																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.589 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.09379 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.167 \text{ kipft}) + ((-0.589 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7782 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8615 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.045 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0071656 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.144 \text{ kipft}) + ((0.045 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.02293 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.341 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8615 \text{ ft}), (1.341 \text{ ft})]$$

$$L_{e,req} = 4.861 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.861 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9722$$

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

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

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

$$q = \frac{(5.772 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36075 \text{ kip/ft}^2$$

$$q = 0.00010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36075 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18038$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.09379 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7782 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7782 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.7782 \text{ kipft/ft})) + (4 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3956 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.7782 \text{ kipft/ft})) + (3 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.7782 \text{ kipft/ft})) + (2 \times (-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.22215 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.7782 \text{ kipft/ft})) + ((-0.09379 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74098 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3956 \text{ ft})}{2}$$

$$p_a = 0.25467 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22215 \text{ kip/ft}^2)}{(0.25467 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8723$$

p_a - Allowable lateral soil pressure at depth L_e ,

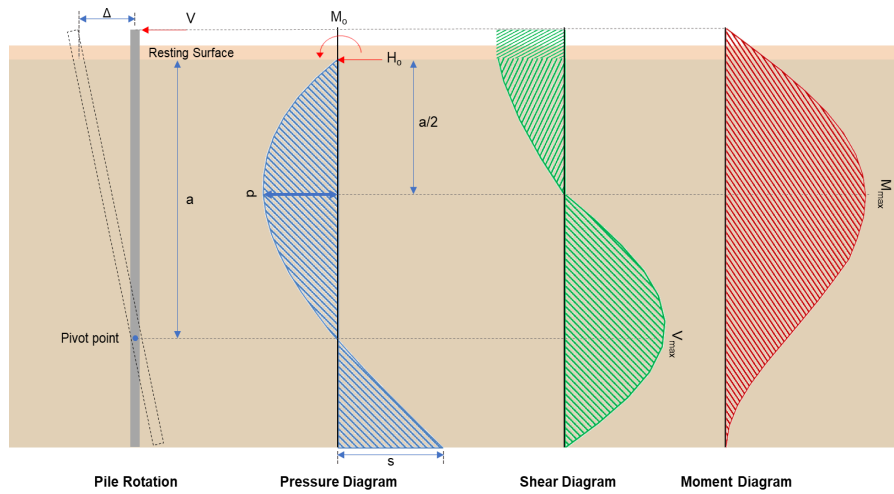
Status: **PASS**
Ratio: **0.870**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74098 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98797$	<p>Status: PASS Ratio: 0.990</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.0071656 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.02293 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.02293 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.0071656 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.02293 \text{ kipft/ft})) + (4 \times (0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5459 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.02293 \text{ kipft/ft})) + (3 \times (0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.02293 \text{ kipft/ft})) + (2 \times (0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.0084763 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.02293 \text{ kipft/ft})) + ((0.0071656 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.019605 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5459 \text{ ft})}{2}$ $p_a = 0.26594 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0084763 \text{ kip/ft}^2)}{(0.26594 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.031873$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.030</p>

$$\text{Ratio} = \frac{(0.019605 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.02614$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.981 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15621 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(19.092 \text{ kipft}) + ((-0.981 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0401 \text{ kipft/ft})}{(-0.15621 \text{ kip/ft})}$$

$$E = 19.462 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0401 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.15621 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.0401 \text{ kipft/ft})) + (4 \times (-0.15621 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3943 \text{ ft}$$

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

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

$$V_{max} = ((-0.15621 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3943 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3943 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.7223 \text{ kip}$$

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

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

$$M_{max} = ((-0.15621 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(19.462 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (19.462 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3943 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.519 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.066 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.01051 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.214 \text{ kipft}) + ((0.066 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.034076 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.034076 \text{ kipft/ft})}{(0.01051 \text{ kip/ft})}$$

$$E = 3.2424 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.034076 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.01051 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.034076 \text{ kipft/ft})) + (4 \times (0.01051 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5445 \text{ ft}$$

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

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

$$V_{max} = ((0.01051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5445 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5445 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.076142 \text{ kip}$$

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

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

$$M_{max} = ((0.01051 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(3.2424 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.2424 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5445 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.17154 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.471 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.315 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.315 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

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

$$\text{Ratio} = \frac{(8.471 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0031665$$

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

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.471 \text{ kip} \rightarrow 8471 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8471 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.61 \text{ kip}$$

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

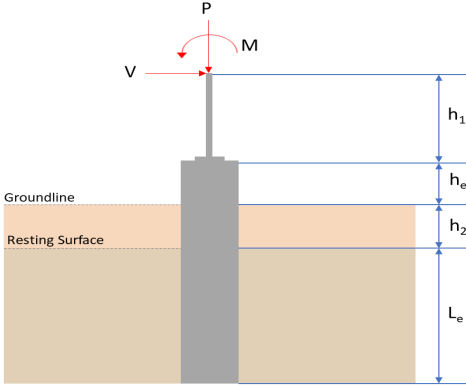
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.61 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.61 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.61 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.83 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.7223 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.7223 \text{ kip})}{(110.83 \text{ kip})}$ $\text{Ratio} = 0.042608$ <p>Considering z-direction:</p> <p>$V_{max} = 0.076142 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.076142 \text{ kip})}{(110.83 \text{ kip})}$ $\text{Ratio} = 0.00068701$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 11.519 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.519 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.046152$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.17154 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.17154 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00068725$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>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 resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.030</td> <td>8.854</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.602</td> <td>-1.008</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.510</td> <td>19.683</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	6.030	8.854	V_x (kip)	-0.602	-1.008	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	11.510	19.683	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.602 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.09586 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(11.51 \text{ kipft}) + ((-0.602 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 1.8328 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 4.9102 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(4.9102 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 4.91 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 5.25 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(4.91 \text{ ft})}{(5.25 \text{ ft})}$ $\text{Ratio} = 0.93524$	<p>Status: PASS Ratio: 0.940</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(6.03 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.37687 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.37687 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.18844$	<p>Status: PASS Ratio: 0.190</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.09586$ kip/ft - Lateral force per length of pile,

$M_o = 1.8328$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8328 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.09586 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.8328 \text{ kipft/ft})) + (4 \times (-0.09586 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5677 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.8328 \text{ kipft/ft})) + (3 \times (-0.09586 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (1.8328 \text{ kipft/ft})) + (2 \times (-0.09586 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20529 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.8328 \text{ kipft/ft})) + ((-0.09586 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.6884 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5677 \text{ ft})}{2}$$

$$p_a = 0.26758 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20529 \text{ kip/ft}^2)}{(0.26758 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76723$$

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

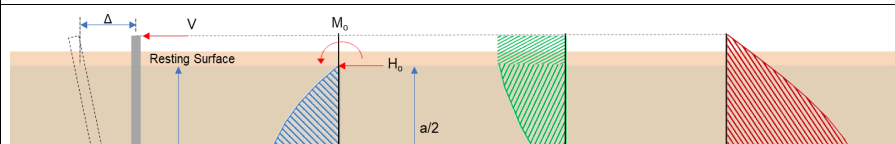
$$\text{Ratio} = \frac{s}{p_s}$$

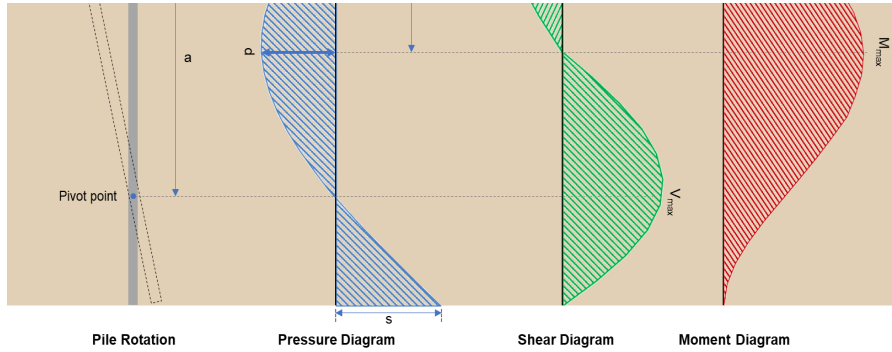
$$\text{Ratio} = \frac{(0.6884 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87416$$

Status: **PASS**
Ratio: **0.770**

Status: **PASS**
Ratio: **0.870**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.008 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.16051 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.683 \text{ kipft}) + ((-1.008 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1342 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.1342 \text{ kipft/ft})}{(-0.16051 \text{ kip/ft})}$$

$$E = 19.527 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1342 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.16051 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.1342 \text{ kipft/ft})) + (4 \times (-0.16051 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5665 \text{ ft}$$

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

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

$$V_{max} = ((-0.16051 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.527 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5665 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.527 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5665 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.655 \text{ kip}$$

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

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

$$M_{max} = ((-0.16051 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(19.527 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5665 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.527 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5665 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.527 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5665 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.911 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.854 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.302 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.302 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

Main reinforcement: **14 - #5 (0.625 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

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

$$\text{Ratio} = \frac{(8.854 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0033097$$

Status: **PASS**
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.854 \text{ kip} \rightarrow 8854 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8854 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.67 \text{ kip}$$

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

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

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

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

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.67 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.67 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.67 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.86 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.655 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.655 \text{ kip})}{(110.86 \text{ kip})}$ $\text{Ratio} = 0.041989$	<p>Status: PASS Ratio: 0.040</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$\phi M_{n,2} = \phi M_{n,1}$

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

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

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:

$M_{max} = 11.911 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.911 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.047719$$

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