

Project Name: test14

Date: Wed Nov 06 2024

Location: 28385 Post Office Rd, Mass City, MI 49948, USA

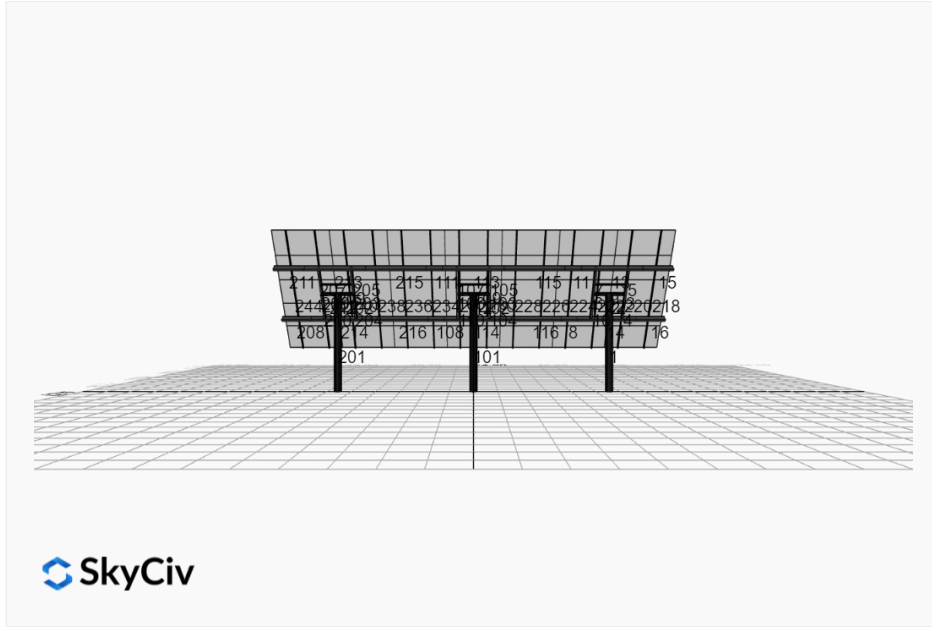
Number of Modules: 35

Unique ID: 3P-17-10TOP-HD-45-L-5Hx7W-IAHC

Number of Poles: 3

Dealer: _____

Date Sold: _____



Array Dimensions N/S	17.42 ft
Array Dimensions E/W	48.42 ft
Winter Tilt Angle	56
Front Edge Clearance	5 ft

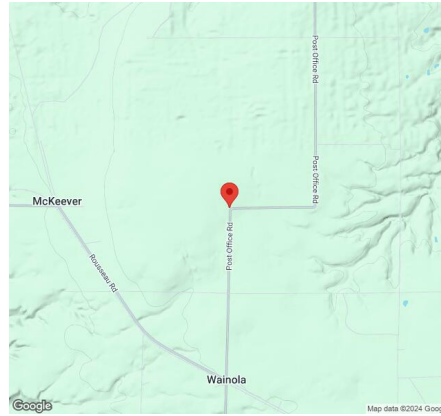
MT Solar Bill of Materials (3P-17-10TOP-HD-45-L-5Hx7W-IAHC)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-57	57IN HD 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:	HD
Module Width:	41.30 in
Module Length:	82.00in
Number of Rows:	5
Number of Columns:	7
Total Number of Modules:	35
Winter Tilt Angle:	56
Front Edge Clearance:	5
Total Array Height at Tilt:	19.44 ft
Total Frame Length:	49.00 ft
Frame Weight:	3795 lbs
Array Dimensions N/S:	17.42 ft
Array Dimensions E/W:	48.42 ft
Rail Length:	209.00 in
Rail Spacing:	3.46 ft

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	12.22 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.00 ft Pile 3: 8.00 ft
Foundation Volume:	14.222 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:	120 mph
Snow Load:	80 psf

Design Disclaimer

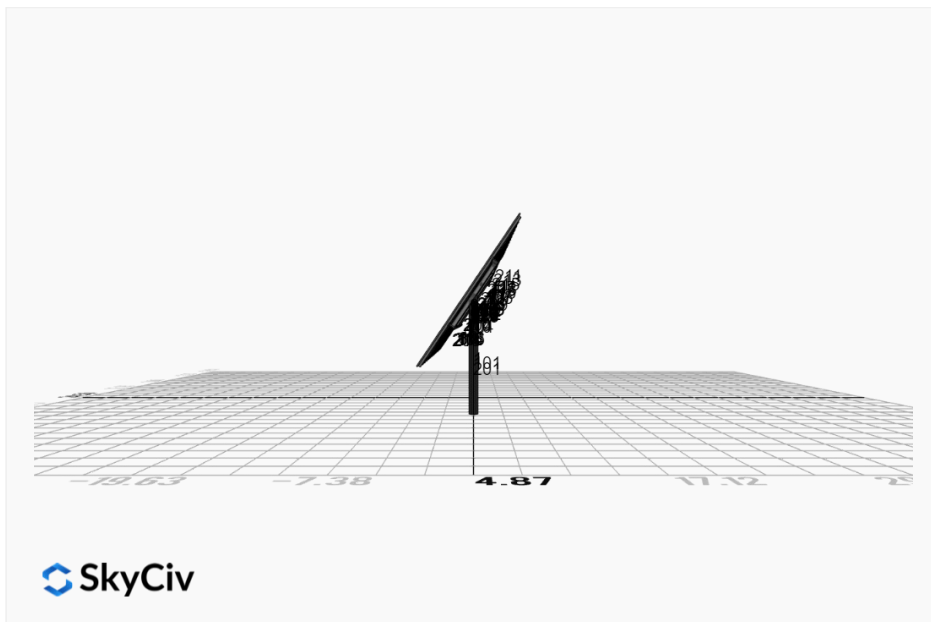
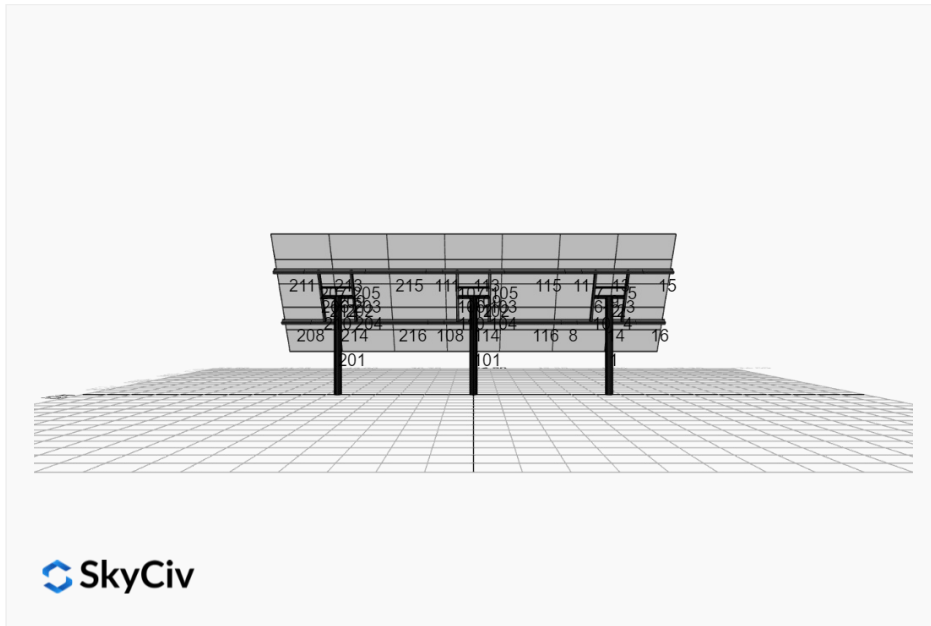
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

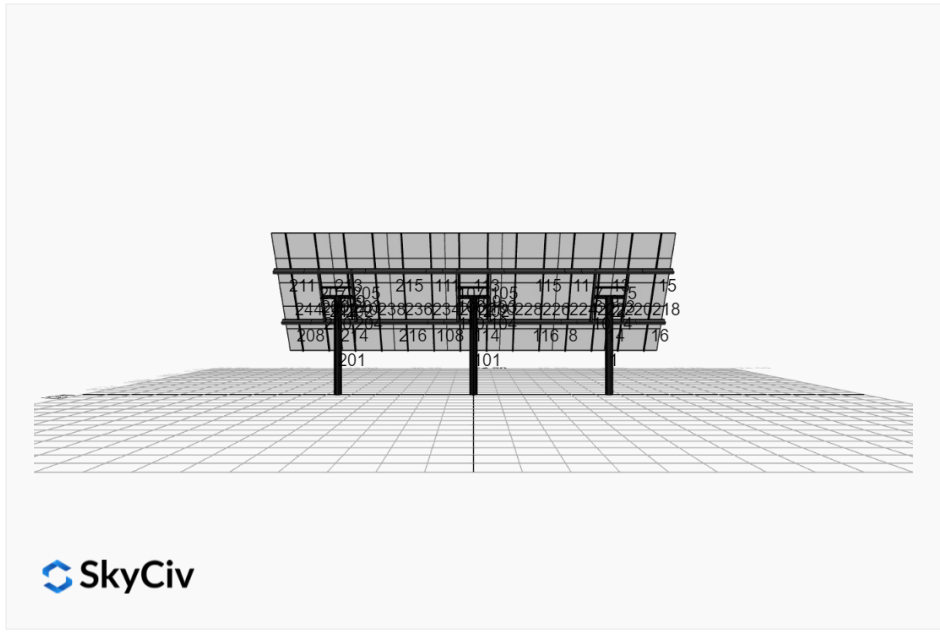
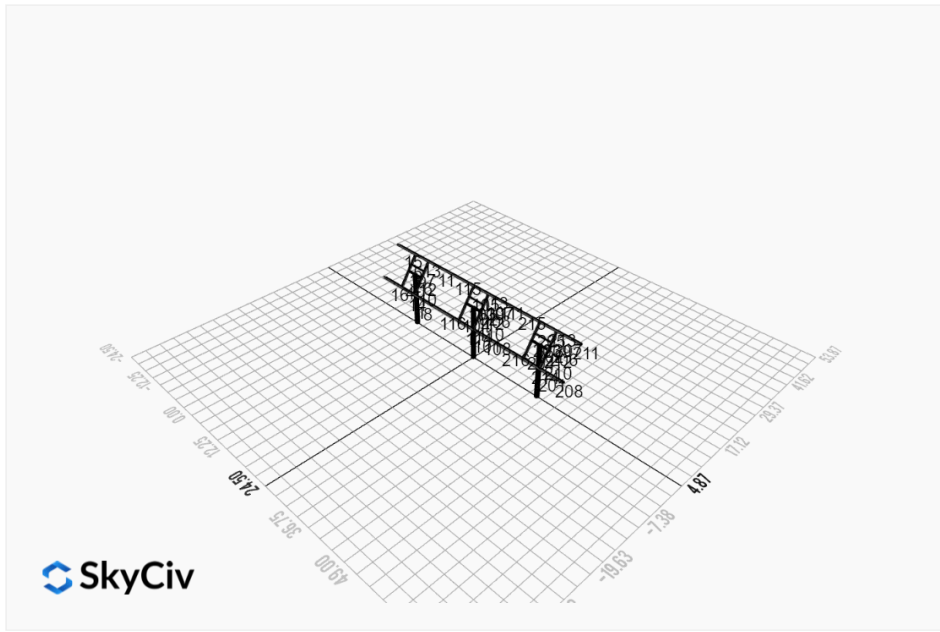
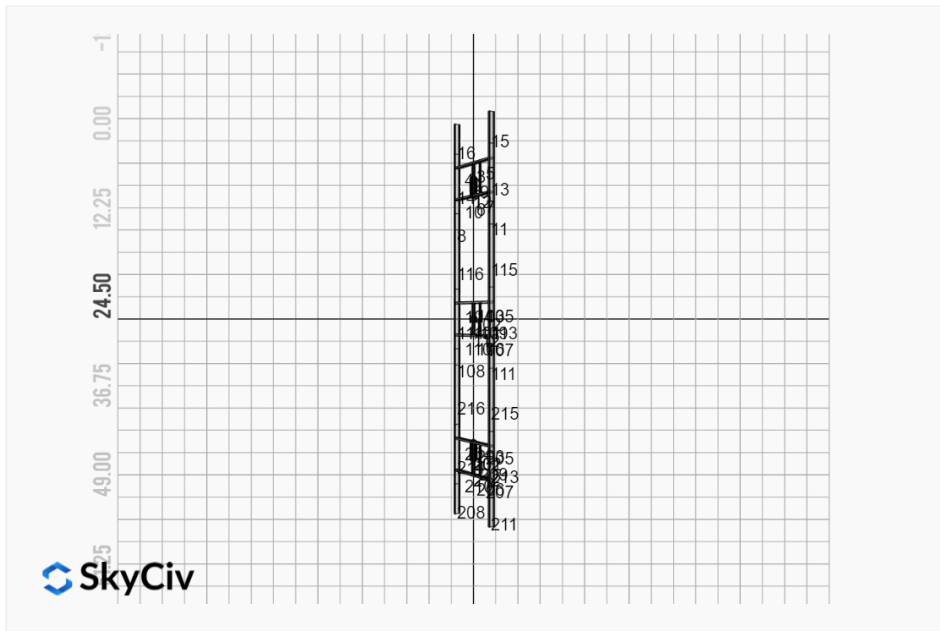
AutoDesigner Input

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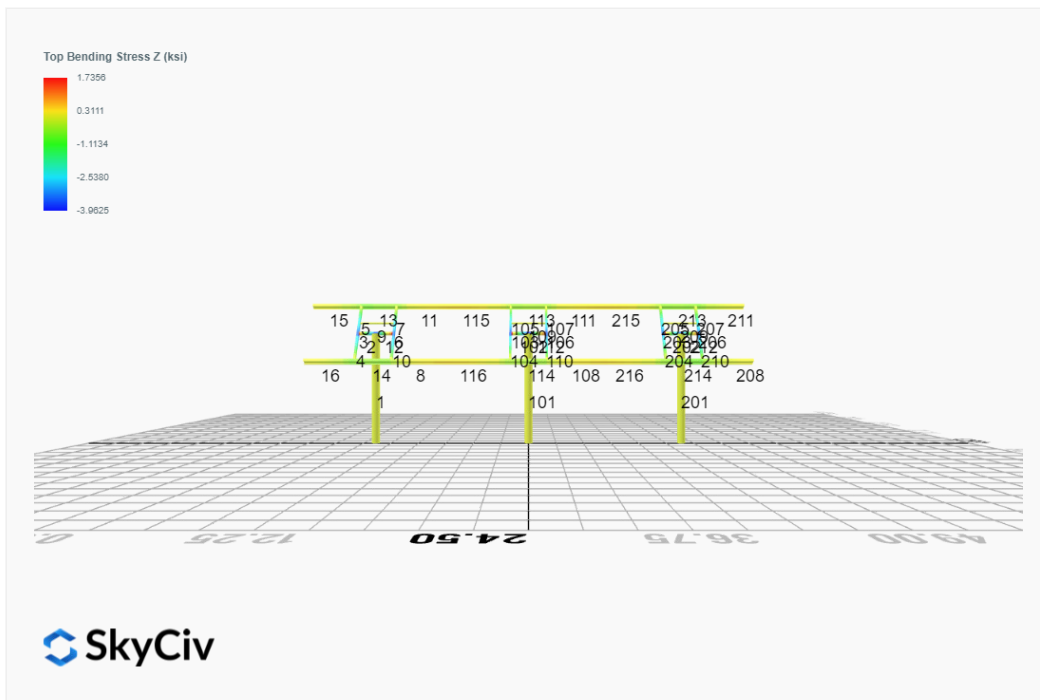
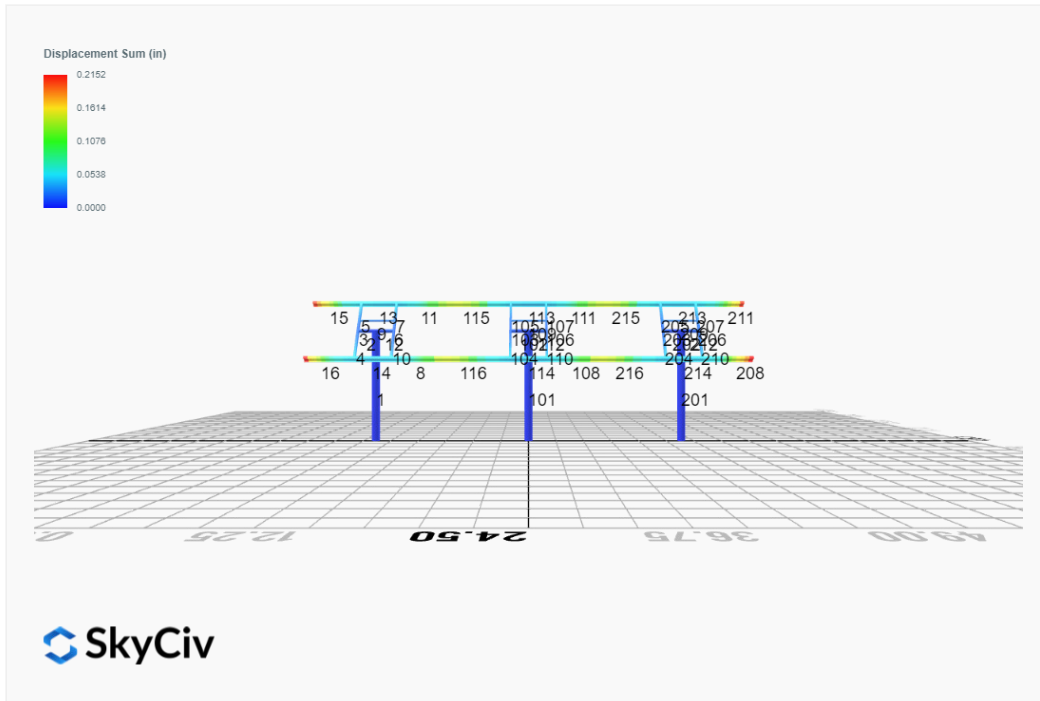
Design Notes:

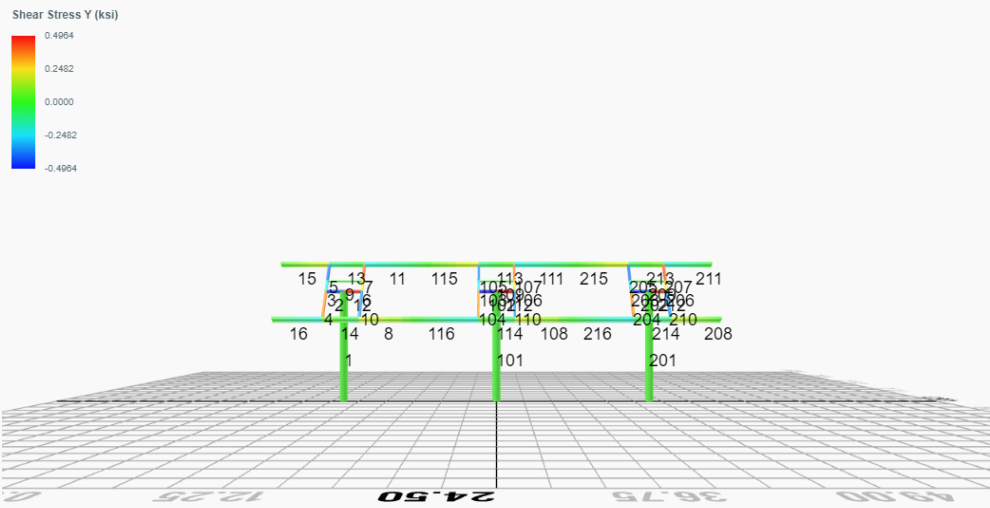
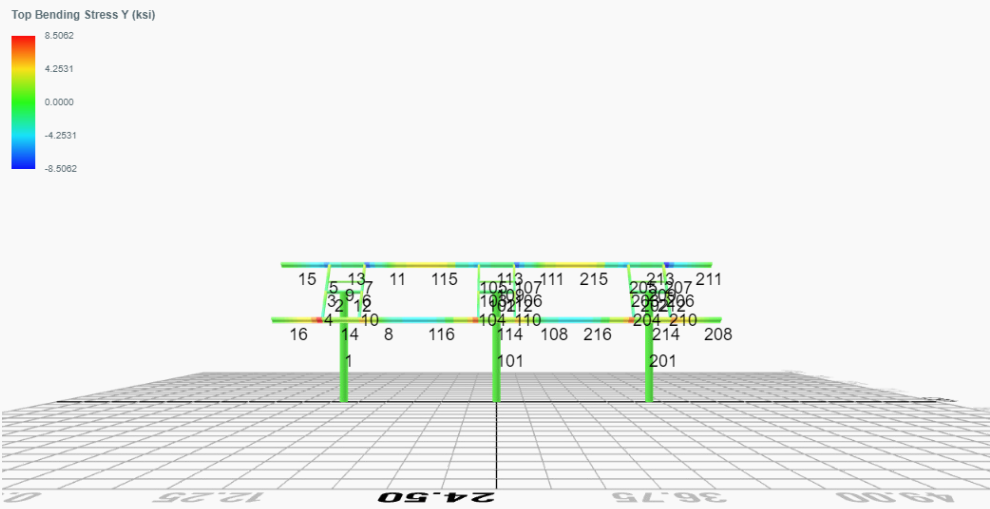
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

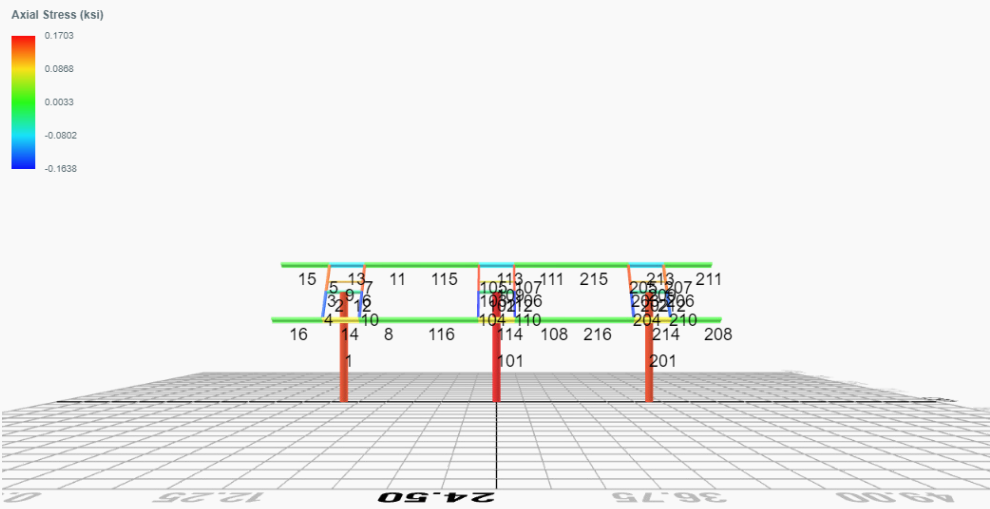




FEM Results (Envelope Worst Case for each member)







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	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 2. D + L	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 3. D + (S or Lr or R)	-0.0130	4.1968	0.0013	0.0109	0.1168	0.1699
ULS: 3. D + (S or Lr or R)	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0112	3.7157	0.0011	0.0093	0.1008	0.1485
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 5b. D + 0.7E	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0112	3.7157	0.0011	0.0093	0.1008	0.1485
ULS: 8. 0.6D + 0.7E	-0.0035	1.3634	0.0003	0.0028	0.0316	0.0507
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5959	5.3642	-0.0096	-0.0260	0.1346	56.6437
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.5837	-0.8195	0.0108	0.0356	-0.0307	-55.5796
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4537	6.0347	-0.0065	-0.0136	0.1622	42.5679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0112	3.7157	0.0011	0.0093	0.1008	0.1485
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4310	1.3969	0.0088	0.0326	0.0382	-41.5996
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0112	3.7157	0.0011	0.0093	0.1008	0.1485
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4484	4.5912	-0.0071	-0.0184	0.1141	42.5039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4363	-0.0466	0.0083	0.0278	-0.0099	-41.6636
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0059	2.2723	0.0005	0.0046	0.0527	0.0845
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5936	4.4553	-0.0098	-0.0278	0.1135	56.6099
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0035	1.3634	0.0003	0.0028	0.0316	0.0507
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.5861	-1.7284	0.0106	0.0337	-0.0518	-55.6134
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0035	1.3634	0.0003	0.0028	0.0316	0.0507

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	8.8424
Shear X	-7.6610
Shear Z	0.0181
Moment X	0.0600
Moment Y (Twist)	0.2335
Moment Z	95.0256

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	6.0347
Shear X	-4.5959
Shear Z	0.0108
Moment X	0.0356
Moment Y (Twist)	0.1622
Moment Z	56.6437

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	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 2. D + L	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 3. D + (S or Lr or R)	0.0260	4.3847	-0.0000	0.0000	0.0000	-0.2719
ULS: 3. D + (S or Lr or R)	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0224	3.8776	-0.0000	0.0000	0.0000	-0.2327

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 5b. D + 0.7E	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0224	3.8776	-0.0000	0.0000	0.0000	-0.2327
ULS: 8. 0.6D + 0.7E	0.0070	1.4138	-0.0000	0.0000	0.0000	-0.0690
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.6632	5.5178	-0.0000	0.0000	0.0000	57.4328
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.6875	-0.8053	-0.0000	0.0000	0.0000	-56.7492
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4838	6.2486	-0.0000	0.0000	0.0000	42.9282
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0224	3.8776	-0.0000	0.0000	0.0000	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.5292	1.5063	-0.0000	0.0000	0.0000	-42.7083
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0224	3.8776	-0.0000	0.0000	0.0000	-0.2327
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4944	4.7274	-0.0000	0.0000	0.0000	43.0459
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.5186	-0.0149	-0.0000	0.0000	0.0000	-42.5906
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0117	2.3564	-0.0000	0.0000	0.0000	-0.1150
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.6679	4.5752	-0.0000	0.0000	0.0000	57.4788
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0070	1.4138	-0.0000	0.0000	0.0000	-0.0690
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.6828	-1.7479	-0.0000	0.0000	0.0000	-56.7032
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0070	1.4138	-0.0000	0.0000	0.0000	-0.0690

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	9.1105
Shear X	-7.8133
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	96.3292

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	6.2486
Shear X	-4.6875
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	57.4788

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	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 2. D + L	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 3. D + (S or Lr or R)	-0.0130	4.1968	-0.0013	-0.0109	-0.1168	0.1699
ULS: 3. D + (S or Lr or R)	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0112	3.7157	-0.0011	-0.0093	-0.1008	0.1485
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 5b. D + 0.7E	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0112	3.7157	-0.0011	-0.0093	-0.1008	0.1485
ULS: 8. 0.6D + 0.7E	-0.0035	1.3634	-0.0003	-0.0028	-0.0316	0.0507
ULS: 5a. D + 0.6W_Wind downforce Case A only	-4.5959	5.3642	0.0096	0.0260	-0.1346	56.6437
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 5a. D + 0.6W_Wind uplift Case A only	4.5837	-0.8195	-0.0108	-0.0356	0.0308	-55.5796
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846

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	-3.4537	6.0347	0.0065	0.0136	-0.1622	42.5679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0112	3.7157	-0.0011	-0.0093	-0.1008	0.1485
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4310	1.3969	-0.0088	-0.0326	-0.0382	-41.5996
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0112	3.7157	-0.0011	-0.0093	-0.1008	0.1485
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.4484	4.5912	0.0071	0.0184	-0.1141	42.5039
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.4363	-0.0466	-0.0083	-0.0278	0.0099	-41.6636
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0059	2.2723	-0.0005	-0.0046	-0.0527	0.0846
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-4.5936	4.4553	0.0098	0.0279	-0.1135	56.6099
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0035	1.3634	-0.0003	-0.0028	-0.0316	0.0507
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	4.5861	-1.7284	-0.0106	-0.0337	0.0518	-55.6134
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0035	1.3634	-0.0003	-0.0028	-0.0316	0.0507

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	8.8424
Shear X	-7.6610
Shear Z	-0.0181
Moment X	-0.0601
Moment Y (Twist)	0.2333
Moment Z	95.0266

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	6.0347
Shear X	-4.5959
Shear Z	-0.0108
Moment X	-0.0356
Moment Y (Twist)	0.1622
Moment Z	56.6437

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: test14
 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)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
11	10in Pipe Sch 40	10.75	0.36				

ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{yD} (in ⁴)	I_{zD} (in ⁴)	I_w (in ⁶)	S_{yD} (in ³)	S_{zD} (in ³)

101	535.87	320.62	147.68	147.68	160.76	160.76
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	24.20	6.12	40.24	43.62
114	133.20	85.85	24.22	6.12	40.24	43.62
115	133.20	86.20	25.08	6.12	40.24	43.62
116	133.20	86.20	25.52	6.12	40.24	43.62
201	535.87	320.62	147.68	147.68	160.76	160.76
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	52.83	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	52.83	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	25.46	6.12	40.24	43.62
214	133.20	85.85	25.46	6.12	40.24	43.62
215	133.20	86.20	26.64	6.12	40.24	43.62
216	133.20	86.20	26.68	6.12	40.24	43.62

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	0.028	0.643	0.001	0.048	0.000	0.658	#13	0.419	Not Required	Pass
2	0.002	0.309	0.333	0.070	0.065	0.643	#13	0.035	Not Required	Pass
3	0.009	0.655	0.061	0.066	0.013	0.695	#13	0.045	Not Required	Pass
4	0.009	0.652	0.126	0.065	0.027	0.703	#13	0.080	Not Required	Pass
5	0.009	0.406	0.128	0.065	0.031	0.422	#13	0.074	Not Required	Pass
6	0.009	0.645	0.064	0.064	0.013	0.685	#13	0.045	Not Required	Pass
7	0.009	0.401	0.124	0.064	0.031	0.419	#13	0.074	Not Required	Pass
8	0.000	0.061	0.085	0.041	0.014	0.127	#21	0.095	Not Required	Pass
9	0.007	0.038	0.061	0.001	0.000	0.101	#13	0.204	Not Required	Pass
10	0.009	0.640	0.120	0.064	0.027	0.696	#13	0.080	Not Required	Pass
11	0.000	0.061	0.084	0.041	0.014	0.125	#21	0.095	Not Required	Pass
12	0.002	0.303	0.322	0.069	0.064	0.626	#13	0.035	Not Required	Pass
13	0.005	0.213	0.317	0.056	0.019	0.451	#21	0.286	Not Required	Pass
14	0.007	0.217	0.317	0.056	0.019	0.452	#21	0.190	Not Required	Pass
15	0.000	0.071	0.147	0.021	0.011	0.106	#21	Not Required	Not Required	Pass

15	0.000	0.071	0.147	0.031	0.011	0.196	#21	Not Required	Not Required	Pass
16	0.000	0.071	0.147	0.031	0.011	0.196	#21	Not Required	Not Required	Pass
101	0.028	0.652	0.000	0.049	0.000	0.667	#13	0.419	Not Required	Pass
102	0.001	0.313	0.329	0.072	0.066	0.642	#13	0.035	Not Required	Pass
103	0.010	0.663	0.070	0.066	0.017	0.708	#13	0.045	Not Required	Pass
104	0.010	0.662	0.116	0.066	0.026	0.716	#13	0.080	Not Required	Pass
105	0.010	0.412	0.118	0.066	0.029	0.428	#13	0.074	Not Required	Pass
106	0.010	0.663	0.070	0.066	0.017	0.708	#13	0.045	Not Required	Pass
107	0.010	0.412	0.118	0.066	0.029	0.428	#13	0.074	Not Required	Pass
108	0.000	0.072	0.086	0.038	0.014	0.133	#21	0.095	Not Required	Pass
109	0.005	0.033	0.056	0.001	0.000	0.091	#13	0.204	Not Required	Pass
110	0.010	0.662	0.116	0.066	0.026	0.716	#13	0.080	Not Required	Pass
111	0.000	0.073	0.085	0.038	0.014	0.132	#21	0.095	Not Required	Pass
112	0.001	0.313	0.329	0.072	0.066	0.642	#13	0.035	Not Required	Pass
113	0.005	0.150	0.283	0.052	0.019	0.374	#21	0.286	Not Required	Pass
114	0.007	0.157	0.282	0.052	0.019	0.374	#21	0.286	Not Required	Pass
115	0.000	0.133	0.160	0.038	0.014	0.250	#21	0.346	Not Required	Pass
116	0.000	0.132	0.161	0.038	0.014	0.251	#21	0.346	Not Required	Pass
201	0.028	0.643	0.001	0.048	0.000	0.658	#13	0.419	Not Required	Pass
202	0.002	0.303	0.322	0.069	0.064	0.626	#13	0.035	Not Required	Pass
203	0.009	0.645	0.064	0.064	0.013	0.685	#13	0.045	Not Required	Pass
204	0.009	0.640	0.120	0.064	0.027	0.696	#13	0.080	Not Required	Pass
205	0.009	0.401	0.124	0.064	0.031	0.419	#13	0.074	Not Required	Pass
206	0.009	0.655	0.061	0.066	0.013	0.695	#13	0.045	Not Required	Pass
207	0.009	0.406	0.128	0.065	0.031	0.422	#13	0.074	Not Required	Pass
208	0.000	0.071	0.147	0.031	0.011	0.196	#21	Not Required	Not Required	Pass
209	0.007	0.038	0.061	0.001	0.000	0.101	#13	0.204	Not Required	Pass
210	0.009	0.652	0.126	0.065	0.027	0.703	#13	0.080	Not Required	Pass
211	0.000	0.071	0.147	0.031	0.011	0.196	#21	Not Required	Not Required	Pass
212	0.002	0.309	0.333	0.070	0.065	0.643	#13	0.035	Not Required	Pass
213	0.005	0.213	0.317	0.056	0.019	0.451	#21	0.190	Not Required	Pass
214	0.007	0.217	0.317	0.056	0.019	0.452	#21	0.286	Not Required	Pass
215	0.000	0.128	0.160	0.041	0.014	0.246	#21	0.346	Not Required	Pass
216	0.000	0.127	0.161	0.041	0.014	0.248	#21	0.346	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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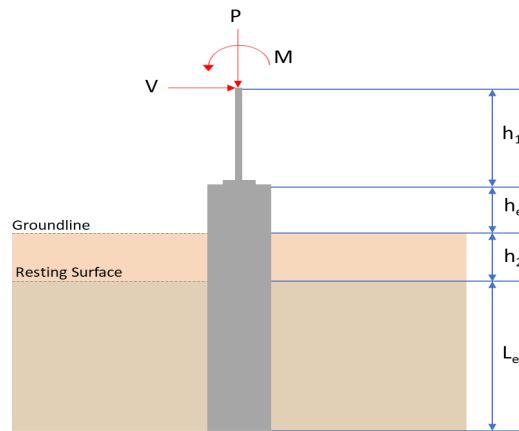
SkyCiv Foundation Design

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

$L = 8$ 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)	6.035	8.842
V_x (kip)	-4.596	-7.661
V_z (kip)	0.011	0.018
M_x (kipft)	0.036	0.060
M_z (kipft)	56.644	95.026

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,

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

$$H_o = \frac{(-4.596 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.73185 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(56.644 \text{ kipft}) + ((-4.596 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.0197 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.3586 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.011 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0017516 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((0.011 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0057325 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.81639 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.3586 \text{ ft}), (0.81639 \text{ ft})]$$

$$L_{e,req} = 7.359 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.359 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.91987$$

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

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

$$q = \frac{(6.035 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37719 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37719 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18859$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.73185 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.0197 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.0197 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (9.0197 \text{ kipft/ft})) + (4 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5347 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.0197 \text{ kipft/ft})) + (3 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (9.0197 \text{ kipft/ft})) + (2 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.26171 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.0197 \text{ kipft/ft})) + ((-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1423 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5347 \text{ ft})}{2}$$

$$p_a = 0.4151 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26171 \text{ kip/ft}^2)}{(0.4151 \text{ kip/ft}^2)}$$

$$Ratio = 0.63046$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1423 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.95193$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.0017516 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0057325 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0057325 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0017516 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.0057325 \text{ kipft/ft})) + (4 \times (0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7465 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0057325 \text{ kipft/ft})) + (3 \times (0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.0057325 \text{ kipft/ft})) + (2 \times (0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.0010938 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0057325 \text{ kipft/ft})) + ((0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.0023885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7465 \text{ ft})}{2}$$

$$p_a = 0.43099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0010938 \text{ kip/ft}^2)}{(0.43099 \text{ kip/ft}^2)}$$

$$Ratio = 0.0025378$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

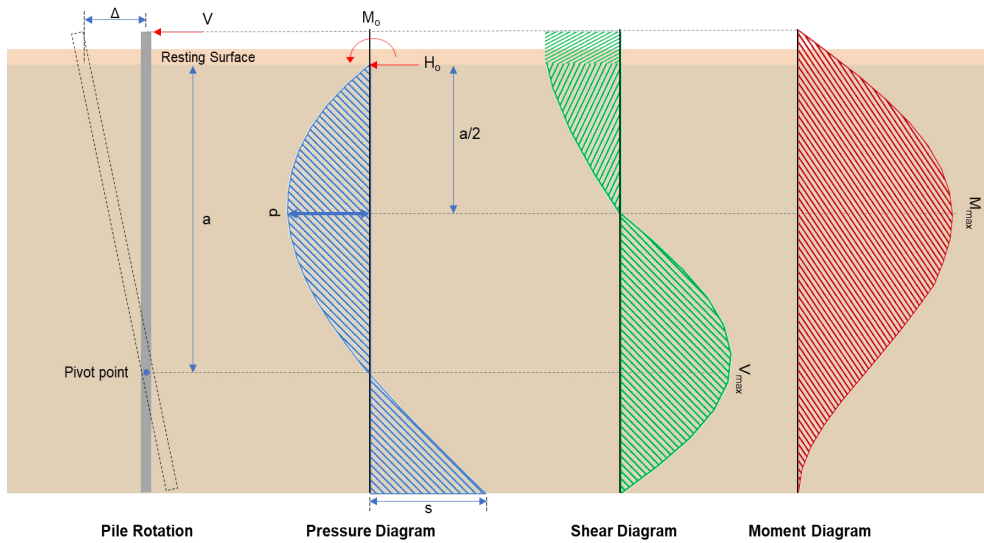
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0023885 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.0019904$$

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,

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

$$H_o = \frac{(-7.661 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2199 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(95.026 \text{ kipft}) + ((-7.661 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.132 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.132 \text{ kipft/ft})}{(-1.2199 \text{ kip/ft})}$$

$$E = 12.404 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.132 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.132 \text{ kipft/ft})) + (4 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.132 \text{ kipft/ft})) + (4 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5338 \text{ ft}$$

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

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

$$V_{max} = ((-1.2199 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5338 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5338 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.605 \text{ kip}$$

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

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

$$M_{max} = ((-1.2199 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.404 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 62.881 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0095541 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0095541 \text{ kipft/ft})}{(0.0028662 \text{ kip/ft})}$$

$$E = 3.3333 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0095541 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0028662 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.0095541 \text{ kipft/ft})) + (4 \times (0.0028662 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7436 \text{ ft}$$

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

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

$$V_{max} = ((0.0028662 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7436 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7436 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.016113 \text{ kip}$$

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

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

$$M_{max} = ((0.0028662 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(3.3333 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.056292 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.842 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(8.842 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0033052$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.842 \text{ kip} \rightarrow 8842 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8842 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

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

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

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

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

Considering x-direction:

V_{max} = 16.605 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.605 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.14978$$

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

Considering z-direction:

$V_{max} = 0.016113 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.016113 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.00014534$$

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

$\phi M_{n,2}$

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

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

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

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:

$M_{max} = 62.881 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(62.881 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.25193$$

Status: **PASS**
Ratio: **0.250**

Considering z-direction:

$M_{max} = 0.056292 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.056292 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00022553$$

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

REFERENCES	CALCULATIONS	RESULTS
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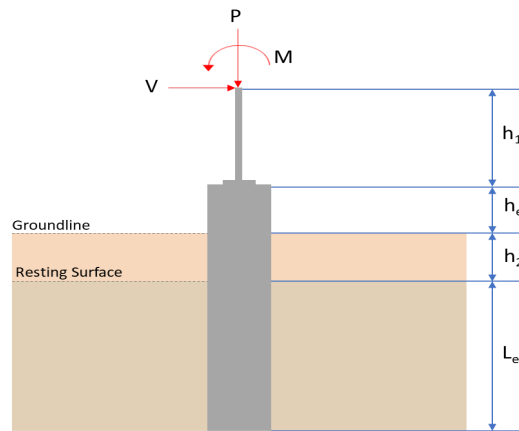
SkyCiv Foundation Design

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

$L = 8$ 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)	6.249	9.110
V_x (kip)	-4.687	-7.813
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	57.479	96.329

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,

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

$$H_o = \frac{(-4.687 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.74634 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.479 \text{ kipft}) + ((-4.687 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.1527 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.379 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(7.379 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.379 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.379 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.92237$$

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

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

$$q = \frac{(6.249 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.39056 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.39056 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19528$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.74634 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.1527 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.1527 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.74634 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (9.1527 \text{ kipft/ft})) + (4 \times (-0.74634 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5354 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.1527 \text{ kipft/ft})) + (3 \times (-0.74634 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (9.1527 \text{ kipft/ft})) + (2 \times (-0.74634 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.26406 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.1527 \text{ kipft/ft})) + ((-0.74634 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1564 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5354 \text{ ft})}{2}$$

$$p_a = 0.41515 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26406 \text{ kip/ft}^2)}{(0.41515 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.63606$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

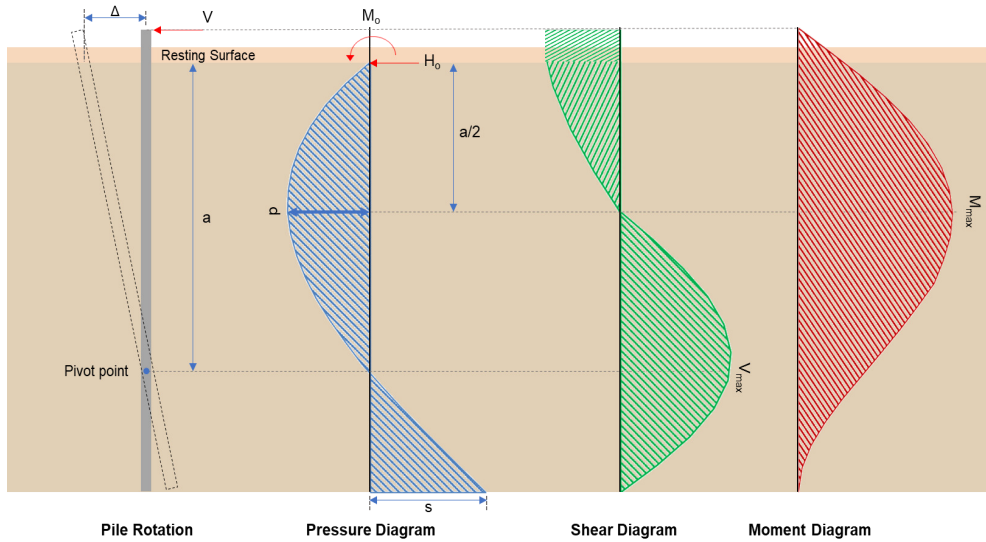
$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1564 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.640**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-7.813 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2441 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(96.329 \text{ kipft}) + ((-7.813 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.339 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.339 \text{ kipft/ft})}{(-1.2441 \text{ kip/ft})}$$

$$E = 12.329 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.339 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.2441 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.339 \text{ kipft/ft})) + (4 \times (-1.2441 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5346 \text{ ft}$$

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

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

$$V_{max} = ((-1.2441 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.329 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5346 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.329 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5346 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.555 \text{ kip}$$

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

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

$$M_{max} = ((-1.2441 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.329 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5346 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.329 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5346 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (12.329 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5346 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 63.801 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{9.11 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.293 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.293 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$Ratio = \frac{(9.11 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0034054$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 b_w = 48 in - Effective width,

d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.11 \text{ kip} \rightarrow 9110 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9110 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.7 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.7 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.7 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.7 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

$V_{max} = 16.853 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.853 \text{ kip})}{(110.89 \text{ kip})}$$

$$\text{Ratio} = 0.15198$$

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

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: $M_{max} = 63.801 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(63.801 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.25561$$

Status: **PASS**
Ratio: **0.260**

REFERENCES	CALCULATIONS	RESULTS
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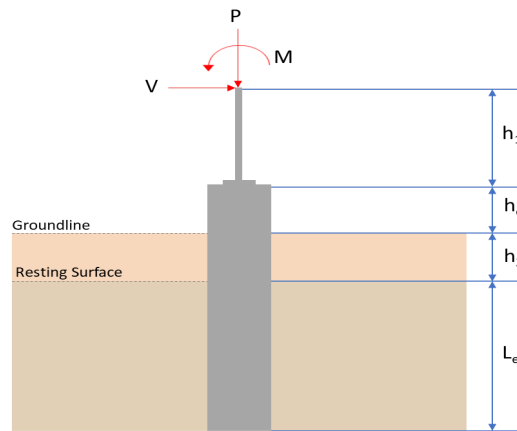
SkyCiv Foundation Design

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

$L = 8$ 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)	6.035	8.842
V_x (kip)	-4.596	-7.661
V_z (kip)	-0.011	-0.018
M_x (kipft)	-0.036	-0.060
M_z (kipft)	56.644	95.027

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,

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

$$H_o = \frac{(-4.596 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.73185 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(56.644 \text{ kipft}) + ((-4.596 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.0197 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.3586 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.011 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0017516 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.036 \text{ kipft}) + ((-0.011 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0057325 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.72571 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(7.3586 \text{ ft}), (0.72571 \text{ ft})]$$

$$L_{e,req} = 7.359 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(7.359 \text{ ft})}{(8 \text{ ft})}$$

$$Ratio = 0.91987$$

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

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

$$q = \frac{(6.035 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37719 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37719 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18859$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.73185 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.0197 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.0197 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (9.0197 \text{ kipft/ft})) + (4 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5347 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.0197 \text{ kipft/ft})) + (3 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (9.0197 \text{ kipft/ft})) + (2 \times (-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.26171 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.0197 \text{ kipft/ft})) + ((-0.73185 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1423 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5347 \text{ ft})}{2}$$

$$p_a = 0.4151 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.26171 \text{ kip/ft}^2)}{(0.4151 \text{ kip/ft}^2)}$$

$$Ratio = 0.63046$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1423 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.95193$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.0017516 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0057325 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0057325 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0017516 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.0057325 \text{ kipft/ft})) + (4 \times (-0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7465 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0057325 \text{ kipft/ft})) + (3 \times (-0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.0057325 \text{ kipft/ft})) + (2 \times (-0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.00039516 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0057325 \text{ kipft/ft})) + ((-0.0017516 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = -0.00023885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7465 \text{ ft})}{2}$$

$$p_a = 0.43099 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00039516 \text{ kip/ft}^2)}{(0.43099 \text{ kip/ft}^2)}$$

$$Ratio = -0.00091688$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8 \text{ ft})$$

$$p_s = 1.2 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

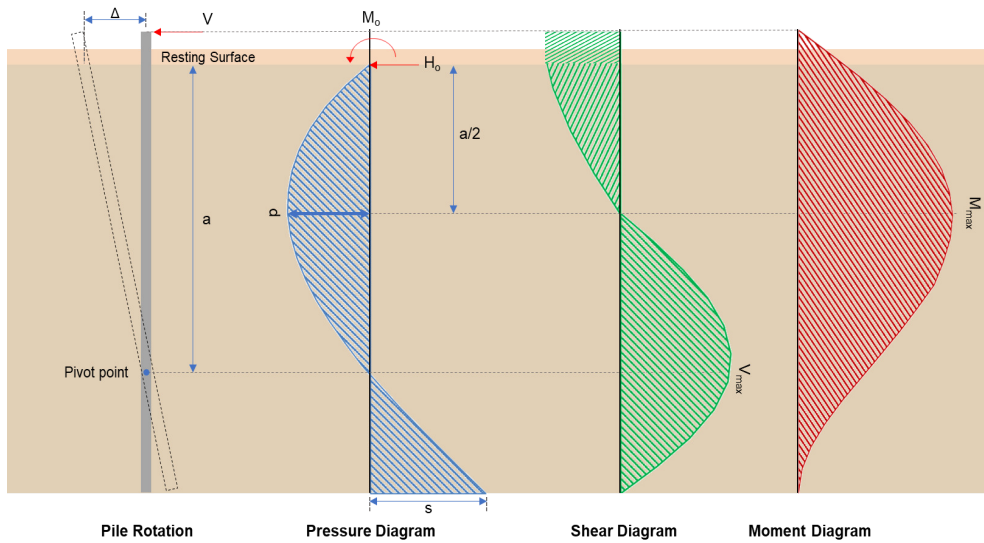
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00023885 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = -0.00019904$$

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,

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

$$H_o = \frac{(-7.661 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.2199 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(95.027 \text{ kipft}) + ((-7.661 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.132 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.132 \text{ kipft/ft})}{(-1.2199 \text{ kip/ft})}$$

$$E = 12.404 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.132 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.132 \text{ kipft/ft})) + (4 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.132 \text{ kipft/ft})) + (4 \times (-1.2199 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5338 \text{ ft}$$

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

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

$$V_{max} = ((-1.2199 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5338 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5338 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 16.605 \text{ kip}$$

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

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

$$M_{max} = ((-1.2199 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(12.404 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.404 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5338 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 62.882 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.018 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0028662 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.06 \text{ kipft}) + ((-0.018 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0095541 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0095541 \text{ kipft/ft})}{(-0.0028662 \text{ kip/ft})}$$

$$E = 3.3333 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0095541 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0028662 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.0095541 \text{ kipft/ft})) + (4 \times (-0.0028662 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7436 \text{ ft}$$

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

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

$$V_{max} = ((-0.0028662 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7436 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7436 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.016113 \text{ kip}$$

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

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

$$M_{max} = ((-0.0028662 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(3.3333 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3333 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7436 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.056292 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.842 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

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

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

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

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

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;">$Ratio = \frac{(8.842 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0033052$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.842 \text{ kip} \rightarrow 8842 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8842 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

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

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

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

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

Considering x-direction:

V_{max} = 16.605 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(16.605 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.14978$$

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

Considering z-direction:

$V_{max} = 0.016113 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.016113 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.00014534$$

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:

$M_{max} = 62.882 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(62.882 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.25193$$

Status: **PASS**
Ratio: **0.250**

Considering z-direction:

$M_{max} = 0.056292 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.056292 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00022553$$

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