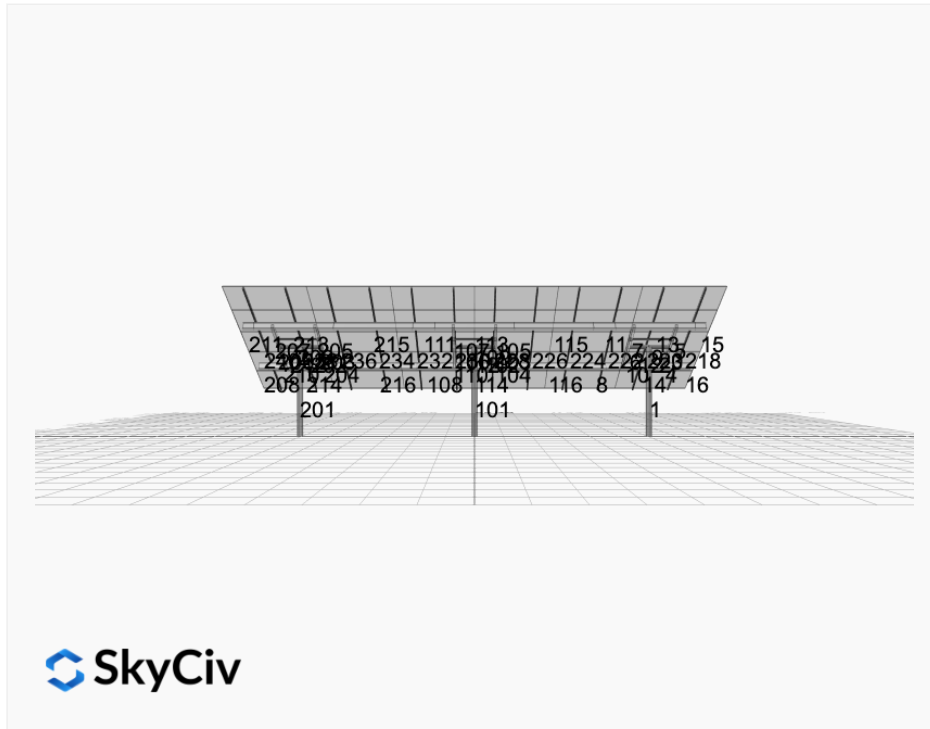


Project Details



Project Name: TBeers 5x6- V1Jb **Date:** Mon Jul 07 2025
Location: Wadsworth, OH 44281, USA **Number of Modules:** 30
Unique ID: 3P-17-6TOP-HD-12-L-5Hx6W-9FA0 **Number of Poles:** 3
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	44.90 ft
Winter Tilt Angle	30
Front Edge Clearance	4 ft

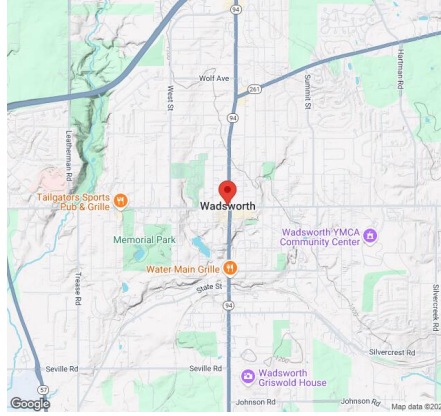
MT Solar Bill of Materials (3P-17-6TOP-HD-12-L-5Hx6W-9FA0)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-12	12IN HD Wing	4
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	6

Rail Bill of Materials

Part	Qty
Rails (226in)	12
Rail Attachment	48
Module Mid Clamp	48
Module End Clamp	24
Ground Lug	6

Site Details:



Site Address: Wadsworth, OH 44281, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	88.80in
Number of Rows:	5
Number of Columns:	6
Total Number of Modules:	30
Winter Tilt Angle:	30
Front Edge Clearance:	4
Total Array Height at Tilt:	13.40 ft
Total Frame Length:	43.50 ft
Module Info/Notes:	
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	44.90 ft
Rail Length:	225.50 in
Rail Spacing:	3.74 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.70 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.50 ft Pile 2: 5.75 ft Pile 3: 5.50 ft
Foundation Volume:	9.926 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Wadsworth, OH 44281, USA
Wind Speed:	102 mph
Snow Load:	20 psf

Design Disclaimer

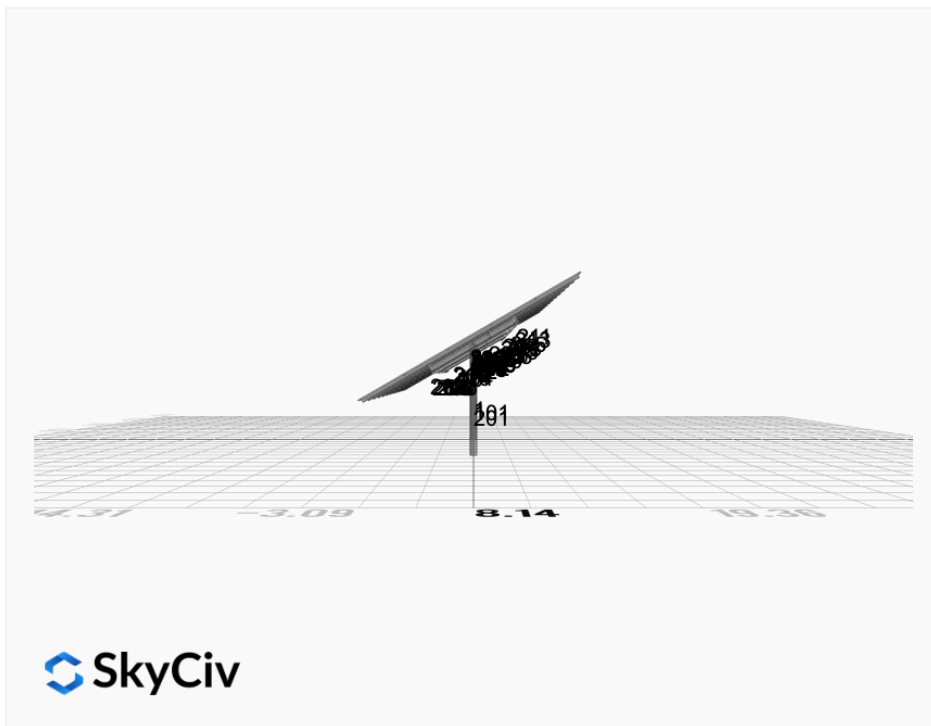
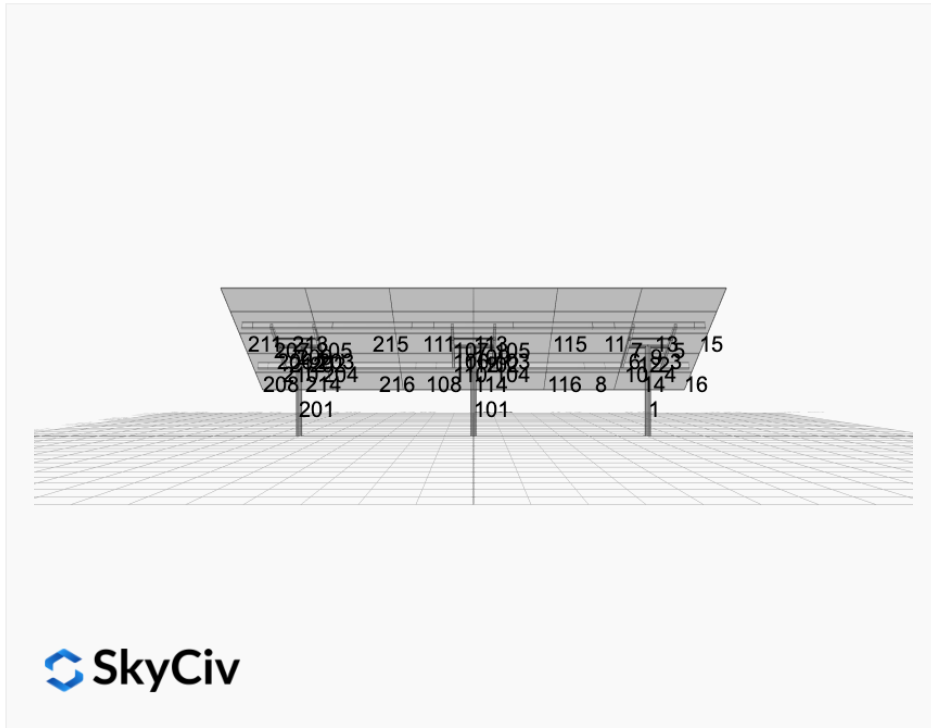
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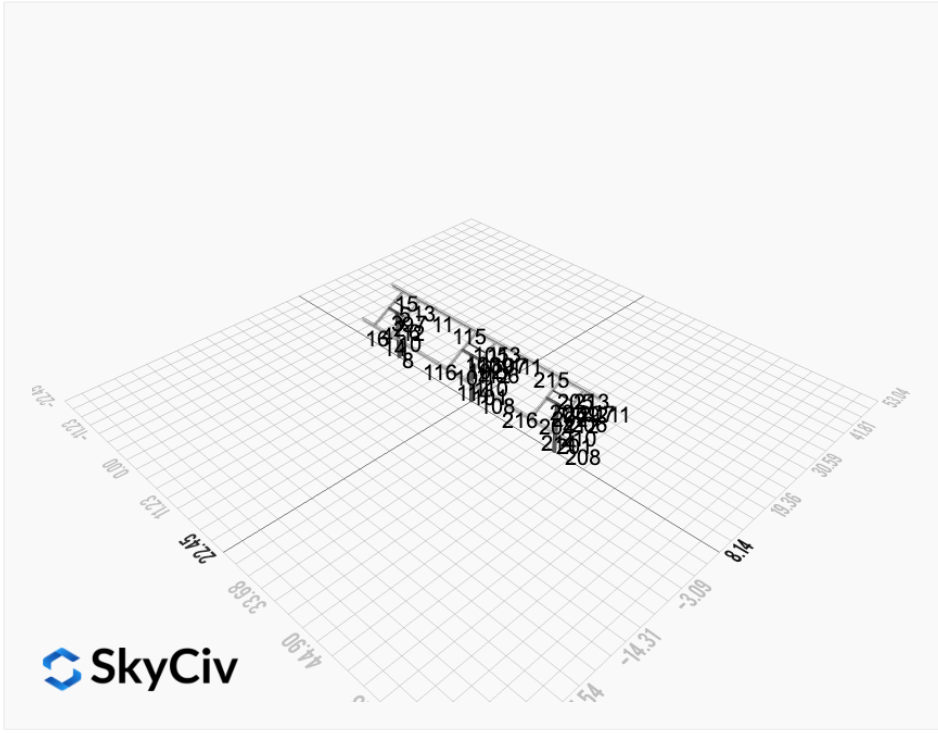
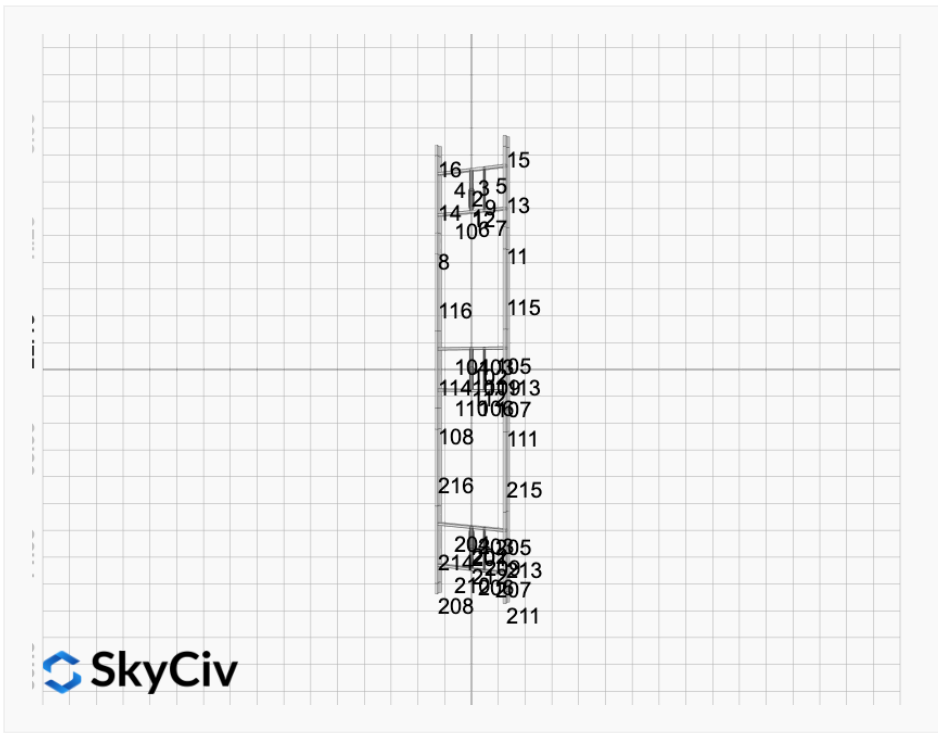
AutoDesigner Input

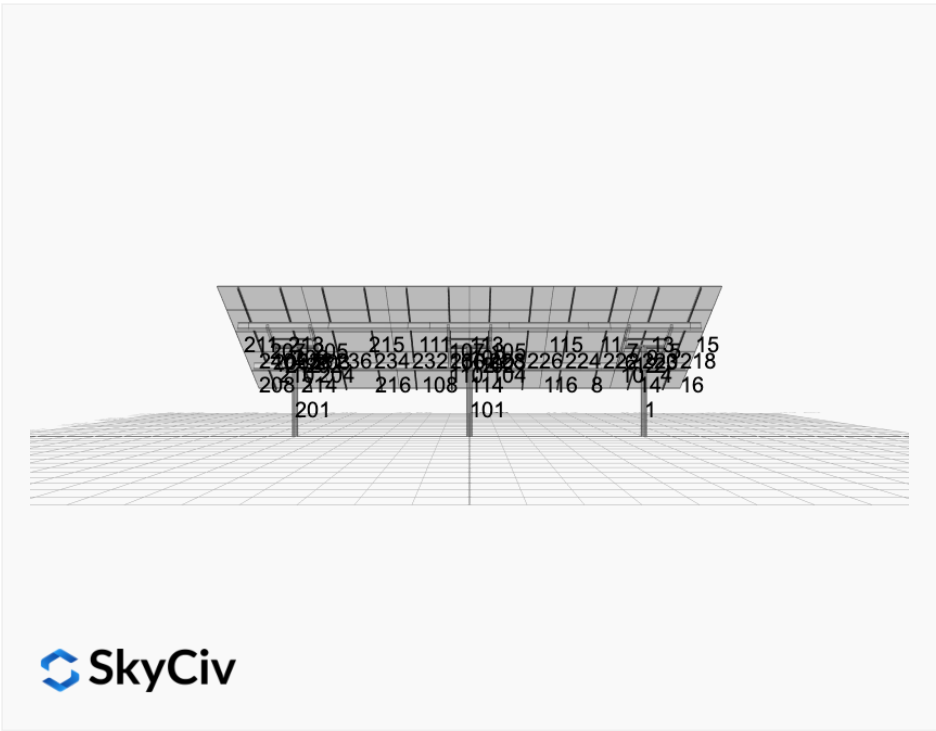
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Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesigner 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)

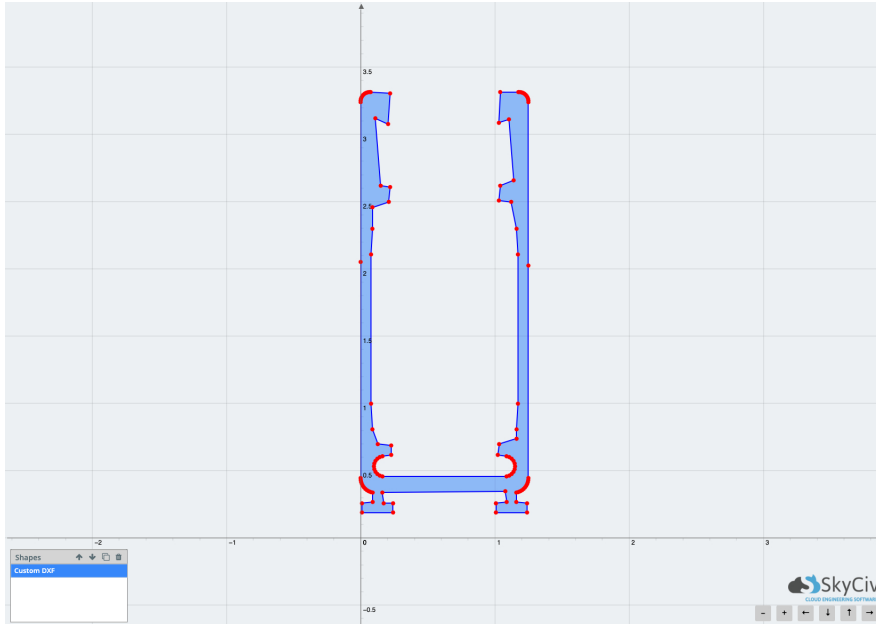






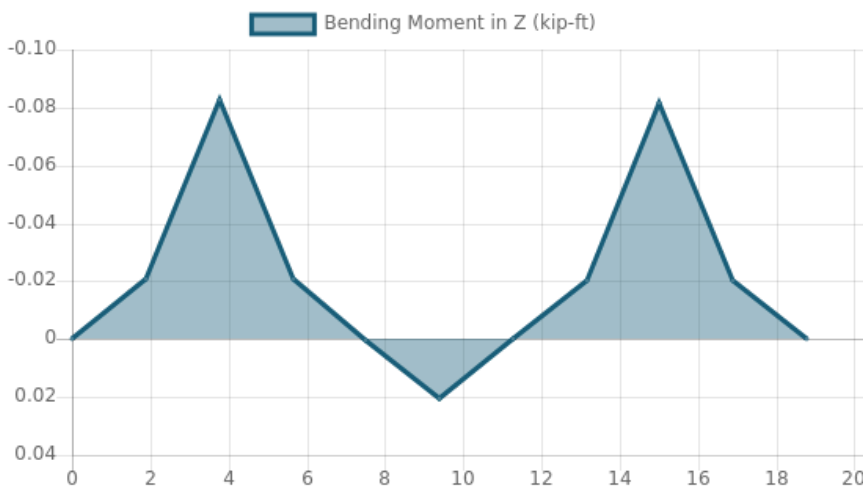
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.741666666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0285 kip/ft
Snow (Y): -0.0165 kip/ft
Wind uplift Case A: 0.0714 kip/ft
Wind uplift Case A: 0.0714 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0992 kip/ft

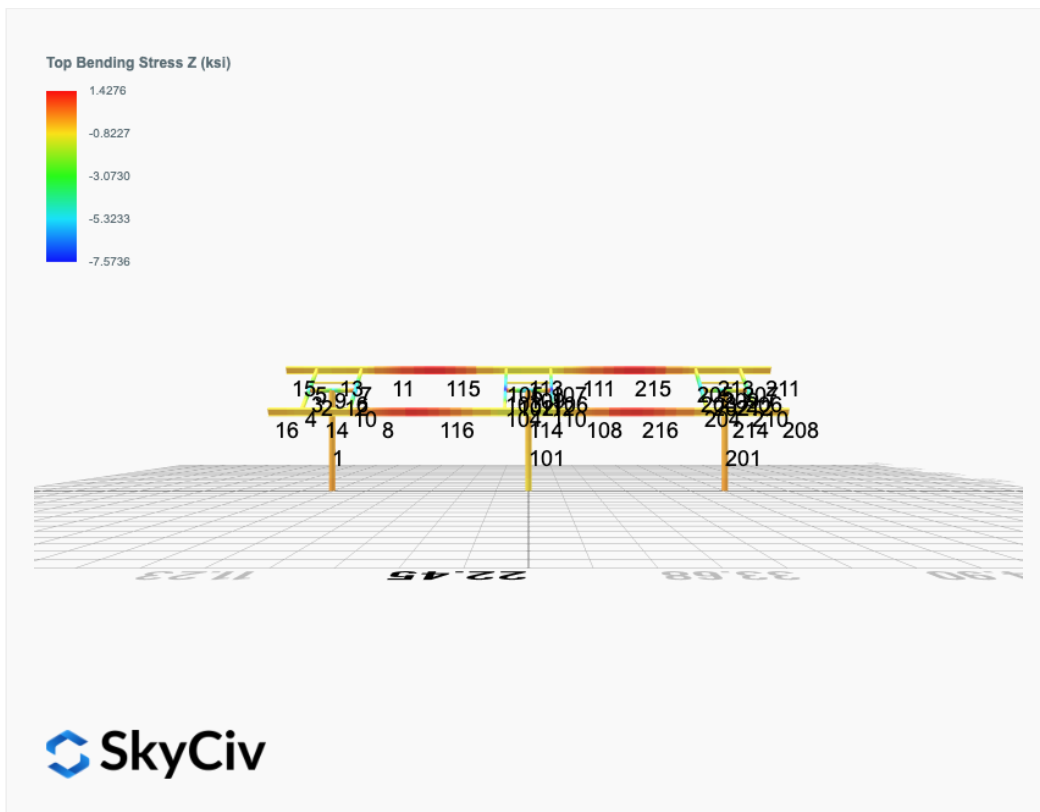
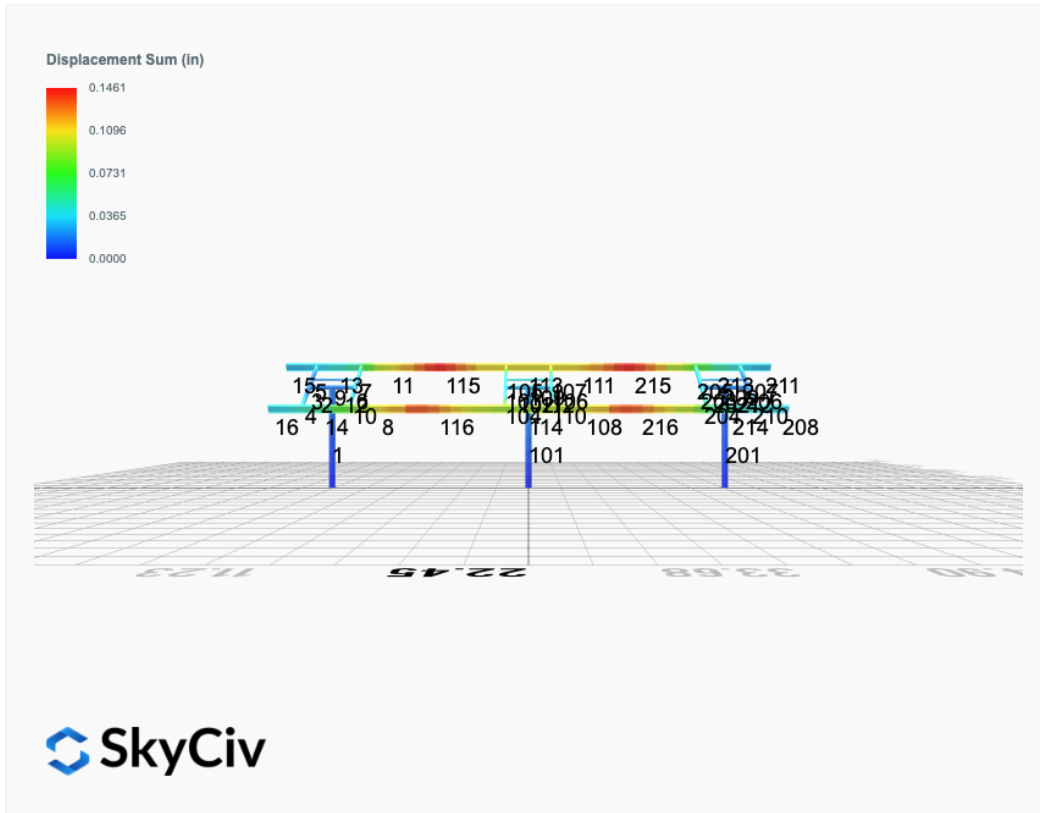


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	15.62556661	0.453	PASS
Material Yield	34.5	15.62556661	0.453	PASS
Material Strength	37	15.62556661	0.422	PASS

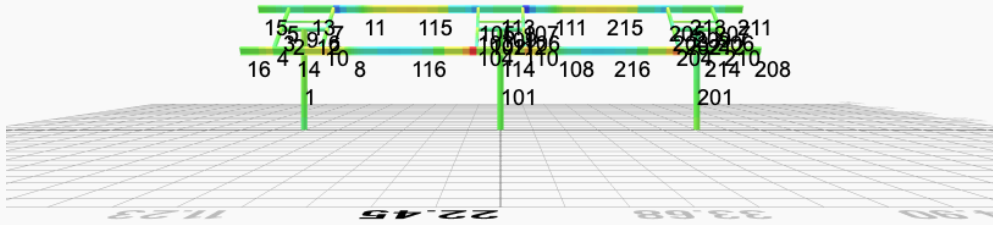
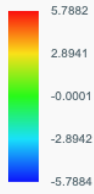
Member 1, ULS: 1. 1.4D



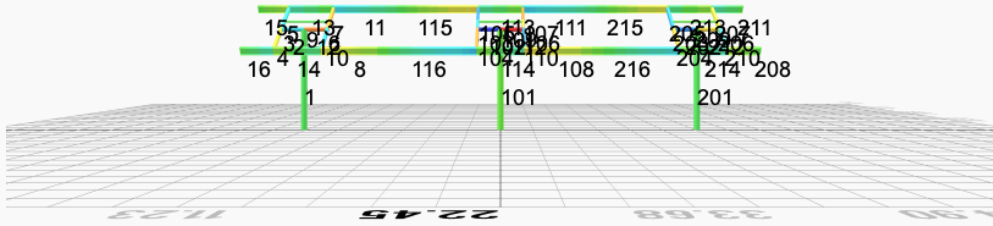
FEM Results (Envelope Worst Case for each member)



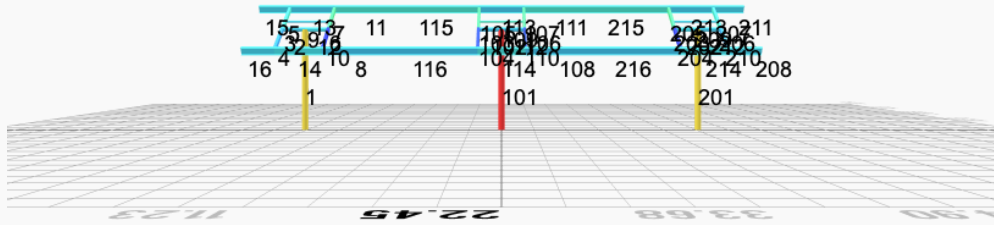
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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	0.0239	1.7267	0.0791	0.2060	-0.0428	-0.1407
ULS: 2. D + L	0.0239	1.7267	0.0791	0.2060	-0.0428	-0.1407
ULS: 3. D + (S or Lr or R)	0.0562	3.5494	0.1862	0.4853	-0.1011	-0.3629
ULS: 3. D + (S or Lr or R)	0.0239	1.7267	0.0791	0.2060	-0.0428	-0.1407
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0481	3.0937	0.1595	0.4155	-0.0865	-0.3074
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0239	1.7267	0.0791	0.2060	-0.0428	-0.1407
ULS: 5b. D + 0.7E	0.0239	1.7267	0.0791	0.2060	-0.0428	-0.1407
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0481	3.0937	0.1595	0.4155	-0.0865	-0.3074
ULS: 8. 0.6D + 0.7E	0.0143	1.0360	0.0475	0.1236	-0.0257	-0.0844
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6763	4.5720	0.3093	0.7865	-0.3911	15.3029
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6763	4.5720	0.3093	0.7865	-0.3911	15.3029
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4762	-0.7087	-0.1132	-0.2775	0.2477	-12.7977
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2666	-0.3219	-0.1053	-0.2570	0.2430	-17.0856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2270	5.2277	0.3321	0.8508	-0.3477	11.2753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2270	5.2277	0.3321	0.8508	-0.3477	11.2753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1374	1.2671	0.0152	0.0529	0.1314	-9.8001
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9802	1.5573	0.0212	0.0682	0.1278	-13.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2513	3.8607	0.2517	0.6414	-0.3040	11.4420
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2513	3.8607	0.2517	0.6414	-0.3040	11.4420
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1131	-0.0999	-0.0651	-0.1566	0.1751	-9.6334
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9560	0.1903	-0.0592	-0.1412	0.1715	-12.8494
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6859	3.8813	0.2777	0.7041	-0.3739	15.3592
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6859	3.8813	0.2777	0.7041	-0.3739	15.3592
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4667	-1.3994	-0.1448	-0.3599	0.2649	-12.7414
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2571	-1.0126	-0.1369	-0.3394	0.2601	-17.0293

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	7.7277
Shear X	-2.8337
Shear Z	0.5356
Moment X	1.3648
Moment Y (Twist)	0.6662
Moment Z	29.0224

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	5.2277
Shear X	-1.6859
Shear Z	0.3321
Moment X	0.8508
Moment Y (Twist)	0.3911
Moment Z	17.0856

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	-0.0478	2.2887	-0.0000	0.0000	0.0000	0.3622
ULS: 2. D + L	-0.0478	2.2887	-0.0000	0.0000	0.0000	0.3622
ULS: 3. D + (S or Lr or R)	-0.1124	4.8711	0.0000	-0.0000	0.0000	0.8232
ULS: 3. D + (S or Lr or R)	-0.0478	2.2887	-0.0000	0.0000	0.0000	0.3622
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0962	4.2255	0.0000	-0.0000	0.0000	0.7079

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0478	2.2887	-0.0000	0.0000	0.0000	0.3622
ULS: 5b. D + 0.7E	-0.0478	2.2887	-0.0000	0.0000	0.0000	0.3622
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0962	4.2255	0.0000	-0.0000	0.0000	0.7079
ULS: 8. 0.6D + 0.7E	-0.0287	1.3732	-0.0000	0.0000	0.0000	0.2173
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2820	6.3577	-0.0000	0.0000	0.0000	19.7246
ULS: 5a. D + 0.6W_Wind downforce Case B only	-2.2820	6.3577	-0.0000	0.0000	0.0000	19.7246
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.8773	-1.2057	-0.0000	0.0000	0.0000	-15.4707
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.4915	-0.5852	-0.0000	0.0000	0.0000	-19.7658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7719	7.2772	0.0000	-0.0000	0.0000	15.2298
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.7719	7.2772	0.0000	-0.0000	0.0000	15.2298
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3476	1.6047	0.0000	-0.0000	0.0000	-11.1667
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.0582	2.0700	0.0000	-0.0000	0.0000	-14.3880
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.7235	5.3405	-0.0000	0.0000	0.0000	14.8840
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.7235	5.3405	-0.0000	0.0000	0.0000	14.8840
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.3960	-0.3321	-0.0000	0.0000	0.0000	-11.5125
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.1067	0.1333	-0.0000	0.0000	0.0000	-14.7338
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2629	5.4422	-0.0000	0.0000	0.0000	19.5798
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-2.2629	5.4422	-0.0000	0.0000	0.0000	19.5798
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.8964	-2.1212	-0.0000	0.0000	0.0000	-15.6156
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.5106	-1.5007	-0.0000	0.0000	0.0000	-19.9106

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.8150
Shear X	-3.8060
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	33.5466

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	7.2772
Shear X	-2.2820
Shear Z	0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	19.9106

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	0.0239	1.7267	-0.0791	-0.2060	0.0428	-0.1407
ULS: 2. D + L	0.0239	1.7267	-0.0791	-0.2060	0.0428	-0.1407
ULS: 3. D + (S or Lr or R)	0.0562	3.5494	-0.1862	-0.4853	0.1011	-0.3629
ULS: 3. D + (S or Lr or R)	0.0239	1.7267	-0.0791	-0.2060	0.0428	-0.1407
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0481	3.0937	-0.1595	-0.4155	0.0865	-0.3074
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0239	1.7267	-0.0791	-0.2060	0.0428	-0.1407
ULS: 5b. D + 0.7E	0.0239	1.7267	-0.0791	-0.2060	0.0428	-0.1407
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0481	3.0937	-0.1595	-0.4155	0.0865	-0.3074
ULS: 8. 0.6D + 0.7E	0.0143	1.0360	-0.0475	-0.1236	0.0257	-0.0844
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6763	4.5720	-0.3093	-0.7865	0.3911	15.3029
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.6763	4.5720	-0.3093	-0.7865	0.3911	15.3029
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4762	-0.7087	0.1132	0.2775	-0.2477	-12.7977
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.2666	-0.3219	0.1053	0.2570	-0.2430	-17.0856

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	-1.2270	5.2277	-0.3321	-0.8508	0.3477	11.2753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2270	5.2277	-0.3321	-0.8508	0.3477	11.2753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1374	1.2671	-0.0152	-0.0529	-0.1314	-9.8001
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9802	1.5573	-0.0212	-0.0682	-0.1278	-13.0160
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2513	3.8607	-0.2517	-0.6414	0.3040	11.4420
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.2513	3.8607	-0.2517	-0.6414	0.3040	11.4420
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1131	-0.0999	0.0651	0.1566	-0.1751	-9.6334
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.9560	0.1903	0.0592	0.1412	-0.1715	-12.8494
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.6859	3.8813	-0.2777	-0.7041	0.3739	15.3592
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.6859	3.8813	-0.2777	-0.7041	0.3739	15.3592
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4667	-1.3994	0.1448	0.3599	-0.2649	-12.7414
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.2571	-1.0126	0.1369	0.3394	-0.2601	-17.0293

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	7.7277
Shear X	-2.8337
Shear Z	-0.5356
Moment X	-1.3648
Moment Y (Twist)	0.6662
Moment Z	29.0232

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	5.2277
Shear X	-1.6859
Shear Z	-0.3321
Moment X	-0.8508
Moment Y (Twist)	0.3911
Moment Z	17.0856

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: TBeers 5x6- V1Jb
 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					
7	6in Pipe Sch 40	6.63	0.28					

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_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

101	251.16	125.14	42.30	42.30	75.35	75.35
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	23.24	6.12	40.24	43.62
114	133.20	85.85	23.01	6.12	40.24	43.62
115	133.20	86.20	24.11	6.12	40.24	43.62
116	133.20	86.20	24.18	6.12	40.24	43.62
201	251.16	125.14	42.30	42.30	75.35	75.35
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	121.82	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	121.82	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	30.77	6.12	40.24	43.62
214	133.20	85.85	31.80	6.12	40.24	43.62
215	133.20	86.20	24.43	6.12	40.24	43.62
216	133.20	86.20	24.36	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	0.062	0.686	0.078	0.037	0.007	0.696	#16	0.488	Not Required	Pass
2	0.001	0.214	0.107	0.052	0.021	0.321	#13	0.053	Not Required	Pass
3	0.004	0.397	0.017	0.039	0.004	0.402	#13	0.045	Not Required	Pass
4	0.003	0.391	0.048	0.039	0.011	0.440	#13	0.080	Not Required	Pass
5	0.004	0.245	0.015	0.039	0.003	0.257	#13	0.074	Not Required	Pass
6	0.007	0.552	0.075	0.057	0.022	0.601	#13	0.045	Not Required	Pass
7	0.007	0.342	0.090	0.055	0.022	0.355	#13	0.074	Not Required	Pass
8	0.003	0.119	0.075	0.030	0.010	0.178	#21	0.095	Not Required	Pass
9	0.002	0.054	0.053	0.003	0.003	0.101	#13	0.204	Not Required	Pass
10	0.008	0.514	0.084	0.052	0.019	0.529	#13	0.080	Not Required	Pass
11	0.004	0.121	0.071	0.034	0.010	0.176	#21	0.095	Not Required	Pass
12	0.002	0.363	0.137	0.074	0.027	0.501	#13	0.053	Not Required	Pass
13	0.005	0.061	0.209	0.047	0.014	0.218	#24	0.286	Not Required	Pass
14	0.004	0.063	0.207	0.043	0.014	0.222	#24	0.190	Not Required	Pass
15	0.000	0.005	0.007	0.008	0.002	0.011	#21	Not Required	Not Required	Pass

15	0.000	0.005	0.007	0.008	0.002	0.011	#21	Not Required	Not Required	Pass
16	0.000	0.005	0.007	0.008	0.002	0.011	#21	Not Required	Not Required	Pass
101	0.086	0.793	0.000	0.051	0.000	0.835	#13	0.488	Not Required	Pass
102	0.001	0.410	0.173	0.089	0.032	0.584	#13	0.053	Not Required	Pass
103	0.007	0.634	0.049	0.064	0.012	0.662	#13	0.045	Not Required	Pass
104	0.007	0.672	0.096	0.067	0.021	0.720	#13	0.080	Not Required	Pass
105	0.007	0.393	0.098	0.063	0.024	0.411	#13	0.074	Not Required	Pass
106	0.007	0.634	0.049	0.064	0.012	0.662	#13	0.045	Not Required	Pass
107	0.007	0.393	0.098	0.063	0.024	0.411	#13	0.074	Not Required	Pass
108	0.003	0.080	0.079	0.042	0.011	0.113	#21	0.095	Not Required	Pass
109	0.006	0.052	0.034	0.001	0.000	0.088	#13	0.204	Not Required	Pass
110	0.007	0.672	0.096	0.067	0.021	0.720	#13	0.080	Not Required	Pass
111	0.004	0.110	0.079	0.037	0.010	0.133	#21	0.095	Not Required	Pass
112	0.001	0.410	0.173	0.089	0.032	0.584	#13	0.053	Not Required	Pass
113	0.005	0.102	0.231	0.050	0.014	0.316	#21	0.286	Not Required	Pass
114	0.007	0.172	0.232	0.055	0.014	0.362	#21	0.286	Not Required	Pass
115	0.005	0.193	0.112	0.037	0.010	0.273	#21	0.346	Not Required	Pass
116	0.004	0.165	0.113	0.042	0.011	0.255	#21	0.346	Not Required	Pass
201	0.062	0.686	0.078	0.037	0.007	0.696	#16	0.488	Not Required	Pass
202	0.002	0.363	0.137	0.074	0.027	0.501	#13	0.053	Not Required	Pass
203	0.007	0.552	0.075	0.057	0.022	0.601	#13	0.045	Not Required	Pass
204	0.008	0.514	0.084	0.052	0.019	0.529	#13	0.080	Not Required	Pass
205	0.007	0.342	0.090	0.055	0.022	0.355	#13	0.074	Not Required	Pass
206	0.004	0.397	0.017	0.039	0.004	0.402	#13	0.045	Not Required	Pass
207	0.004	0.245	0.014	0.039	0.003	0.257	#13	0.074	Not Required	Pass
208	0.000	0.005	0.007	0.008	0.002	0.011	#21	Not Required	Not Required	Pass
209	0.002	0.054	0.053	0.003	0.003	0.101	#13	0.204	Not Required	Pass
210	0.003	0.391	0.048	0.039	0.011	0.440	#13	0.120	Not Required	Pass
211	0.000	0.005	0.007	0.008	0.002	0.011	#21	Not Required	Not Required	Pass
212	0.001	0.214	0.107	0.052	0.021	0.321	#13	0.053	Not Required	Pass
213	0.005	0.061	0.209	0.047	0.014	0.218	#24	0.190	Not Required	Pass
214	0.004	0.063	0.207	0.043	0.014	0.222	#24	0.286	Not Required	Pass
215	0.005	0.198	0.112	0.034	0.010	0.281	#21	0.346	Not Required	Pass
216	0.004	0.182	0.113	0.030	0.010	0.268	#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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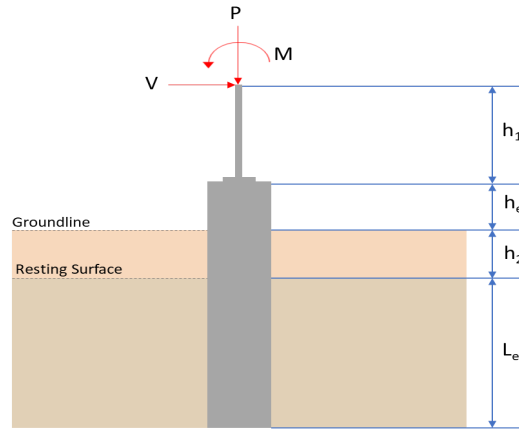
SkyCiv Foundation Design

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

$L = 5.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.228	7.728
V_x (kip)	-1.686	-2.834
V_z (kip)	0.332	0.536
M_x (kipft)	0.851	1.365
M_z (kipft)	17.086	29.022

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,

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

$$H_o = \frac{(-1.686 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.26847 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.086 \text{ kipft}) + ((-1.686 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7207 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1302 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.332 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.052866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.851 \text{ kipft}) + ((0.332 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13551 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.6851 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.1302 \text{ ft}), (2.6851 \text{ ft})]$$

$$L_{e,req} = 5.13 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(5.13 \text{ ft})}{(5.5 \text{ ft})}$$

$$Ratio = 0.93273$$

Status: **PASS**
Ratio: **0.930**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.228 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.32675 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.32675 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16338$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26847 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7207 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7207 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7207 \text{ kipft/ft})) + (4 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7884 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.7207 \text{ kipft/ft})) + (3 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.7207 \text{ kipft/ft})) + (2 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.19821 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.7207 \text{ kipft/ft})) + ((-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.78641 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7884 \text{ ft})}{2}$$

$$p_a = 0.28413 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19821 \text{ kip/ft}^2)}{(0.28413 \text{ kip/ft}^2)}$$

$$Ratio = 0.69758$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.78641 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.95322$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = 0.052866 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.13551 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13551 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.052866 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.13551 \text{ kipft/ft})) + (4 \times (0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9364 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.13551 \text{ kipft/ft})) + (3 \times (0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.13551 \text{ kipft/ft})) + (2 \times (0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.050194 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.13551 \text{ kipft/ft})) + ((0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.11143 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9364 \text{ ft})}{2}$$

$$p_a = 0.29523 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.050194 \text{ kip/ft}^2)}{(0.29523 \text{ kip/ft}^2)}$$

$$Ratio = 0.17002$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.11143 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.13506$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.834 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45127 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.022 \text{ kipft}) + ((-2.834 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.6213 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.6213 \text{ kipft/ft})}{(-0.45127 \text{ kip/ft})}$$

$$E = 10.241 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.6213 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times 4.6213) + (4 \times (-0.45127) \times 5.5)}$$

$$a = \frac{(-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft})}{(6 \times (4.6213 \text{ kipft/ft})) + (4 \times (-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7875 \text{ ft}$$

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

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

$$V_{max} = ((-0.45127 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7875 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7875 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.1383 \text{ kip}$$

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

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

$$M_{max} = ((-0.45127 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.241 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7875 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7875 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7875 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 18.728 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.536 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.08535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.365 \text{ kipft}) + ((0.536 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21736 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21736 \text{ kipft/ft})}{(0.08535 \text{ kip/ft})}$$

$$E = 2.5466 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21736 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (0.08535 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.21736 \text{ kipft/ft})) + (4 \times (0.08535 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9371 \text{ ft}$$

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

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

$$V_{max} = ((0.08535 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9371 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9371 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.50745 \text{ kip}$$

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

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

$$M_{max} = ((0.08535 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.5466 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2282 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.728 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.339 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.339 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(7.728 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0028888$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.728 \text{ kip} \rightarrow 7728 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7728 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.52 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.52 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.52 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.52 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.77 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.1383 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.1383 \text{ kip})}{(110.77 \text{ kip})}$$

$$Ratio = 0.064445$$

Considering z-direction:

$V_{max} = 0.50745 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.50745 \text{ kip})}{(110.77 \text{ kip})}$$

$$Ratio = 0.0045813$$

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

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:

$M_{max} = 18.728 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.728 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.075033$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$M_{max} = 1.2282 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.2282 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0049206$$

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

REFERENCES	CALCULATIONS	RESULTS
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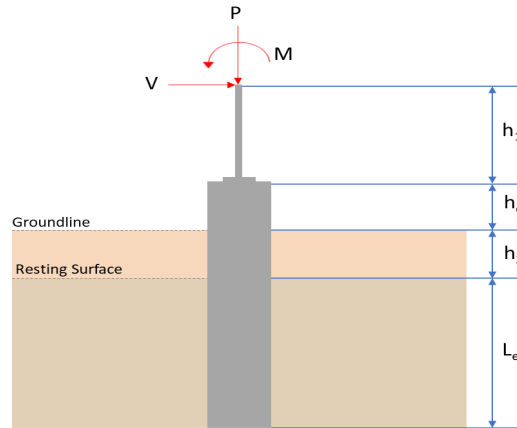
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.228	7.728
V_x (kip)	-1.686	-2.834
V_z (kip)	-0.332	-0.536
M_x (kipft)	-0.851	-1.365
M_z (kipft)	17.086	29.023

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,

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

$$H_o = \frac{(-1.686 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.26847 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.086 \text{ kipft}) + ((-1.686 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.7207 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1302 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.332 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.052866 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.851 \text{ kipft}) + ((-0.332 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13551 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7443 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1302 \text{ ft}), (1.7443 \text{ ft})]$$

$$L_{e,req} = 5.13 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.13 \text{ ft})}{(5.5 \text{ ft})}$$

$$\text{Ratio} = 0.93273$$

Status: **PASS**
Ratio: **0.930**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.228 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.32675 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.32675 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.16338$$

Status: **PASS**
Ratio: **0.160**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.26847 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7207 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7207 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (2.7207 \text{ kipft/ft})) + (4 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7884 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.7207 \text{ kipft/ft})) + (3 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (2.7207 \text{ kipft/ft})) + (2 \times (-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = 0.19821 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.7207 \text{ kipft/ft})) + ((-0.26847 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = 0.78641 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7884 \text{ ft})}{2}$$

$$p_a = 0.28413 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19821 \text{ kip/ft}^2)}{(0.28413 \text{ kip/ft}^2)}$$

$$Ratio = 0.69758$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.78641 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = 0.95322$$

Status: **PASS**
Ratio: **0.700**

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

Considering z-direction:

$H_o = -0.052866 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.13551 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13551 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.052866 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.13551 \text{ kipft/ft})) + (4 \times (-0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9364 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.13551 \text{ kipft/ft})) + (3 \times (-0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]^2}{(5.5 \text{ ft})^2 \times [(3 \times (0.13551 \text{ kipft/ft})) + (2 \times (-0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]}$$

$$p = -0.015452 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.13551 \text{ kipft/ft})) + ((-0.052866 \text{ kip/ft}) \times (5.5 \text{ ft}))]}{(5.5 \text{ ft})^2}$$

$$s = -0.0039164 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9364 \text{ ft})}{2}$$

$$p_a = 0.29523 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.015452 \text{ kip/ft}^2)}{(0.29523 \text{ kip/ft}^2)}$$

$$Ratio = -0.05234$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.5 \text{ ft})$$

$$p_s = 0.825 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0039164 \text{ kip/ft}^2)}{(0.825 \text{ kip/ft}^2)}$$

$$Ratio = -0.0047472$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.834 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45127 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.023 \text{ kipft}) + ((-2.834 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.6215 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.6215 \text{ kipft/ft})}{(-0.45127 \text{ kip/ft})}$$

$$E = 10.241 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.6215 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (4.6215 \text{ kipft/ft})) + (4 \times (-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = \frac{(-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft})}{(6 \times (4.6215 \text{ kipft/ft})) + (4 \times (-0.45127 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.7875 \text{ ft}$$

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

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

$$V_{max} = ((-0.45127 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7875 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7875 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.1385 \text{ kip}$$

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

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

$$M_{max} = ((-0.45127 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(10.241 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.7875 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.7875 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (10.241 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.7875 \text{ ft})}{(2 \times (5.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 18.729 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.536 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.08535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.365 \text{ kipft}) + ((-0.536 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21736 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21736 \text{ kipft/ft})}{(-0.08535 \text{ kip/ft})}$$

$$E = 2.5466 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21736 \text{ kipft/ft}) \times (5.5 \text{ ft})) + (3 \times (-0.08535 \text{ kip/ft}) \times (5.5 \text{ ft})^2)}{(6 \times (0.21736 \text{ kipft/ft})) + (4 \times (-0.08535 \text{ kip/ft}) \times (5.5 \text{ ft}))}$$

$$a = 3.9371 \text{ ft}$$

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

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

$$V_{max} = ((-0.08535 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9371 \text{ ft})}{(5.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9371 \text{ ft})}{(5.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.50745 \text{ kip}$$

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

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

$$M_{max} = ((-0.08535 \text{ kip/ft}) \times (48 \text{ in}) \times (5.5 \text{ ft})) \times \left[\left(\frac{(2.5466 \text{ ft})}{(5.5 \text{ ft})} + \frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.5466 \text{ ft})}{(5.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.9371 \text{ ft})}{2 \times (5.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2282 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.728 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.339 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.339 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(7.728 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0028888$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.728 \text{ kip} \rightarrow 7728 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7728 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.52 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.52 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.52 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.52 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.77 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.1385 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.1385 \text{ kip})}{(110.77 \text{ kip})}$$

$$Ratio = 0.064447$$

Considering z-direction:

$V_{max} = 0.50745 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.50745 \text{ kip})}{(110.77 \text{ kip})}$$

$$Ratio = 0.0045813$$

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

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:

$M_{max} = 18.729 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.729 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.075035$$

Status: **PASS**
Ratio: **0.080**

Considering z-direction:

$M_{max} = 1.2282 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(1.2282 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0049206$$

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

REFERENCES	CALCULATIONS	RESULTS
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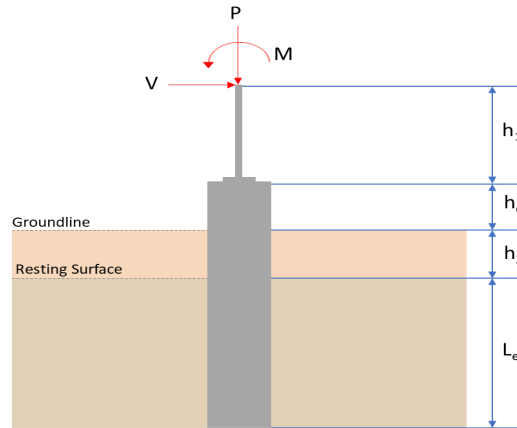
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	7.277	10.815
V_x (kip)	-2.282	-3.806
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	19.911	33.547

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.282 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36338 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.911 \text{ kipft}) + ((-2.282 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.1705 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1967 \text{ ft}$ - 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,

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

$$L_{e,req} = \text{MAX}[(5.1967 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.197 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.197 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.90383$$

Status: **PASS**
Ratio: **0.900**

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

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

$$q = \frac{(7.277 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.45481 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.45481 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22741$$

Status: **PASS**
Ratio: **0.230**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.36338 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.1705 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.1705 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.36338 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.1705 \text{ kipft/ft})) + (4 \times (-0.36338 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9796 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.1705 \text{ kipft/ft})) + (3 \times (-0.36338 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.1705 \text{ kipft/ft})) + (2 \times (-0.36338 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.17499 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.1705 \text{ kipft/ft})) + ((-0.36338 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.77157 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9796 \text{ ft})}{2}$$

$$p_a = 0.29847 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.17499 \text{ kip/ft}^2)}{(0.29847 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.5863$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

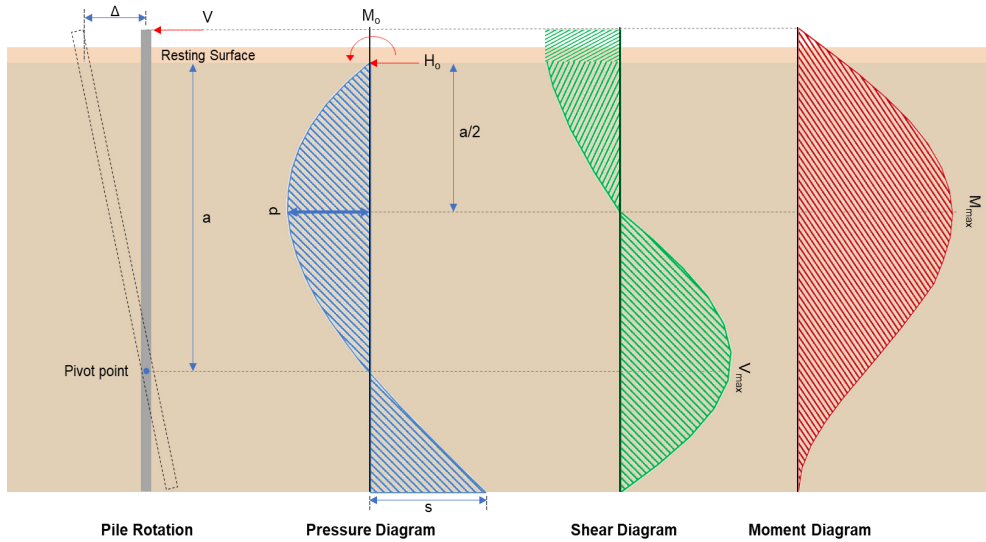
$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.77157 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.590**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.806 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(33.547 \text{ kipft}) + ((-3.806 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.3419 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.3419 \text{ kipft/ft})}{(-0.60605 \text{ kip/ft})}$$

$$E = 8.8142 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.3419 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.60605 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.3419 \text{ kipft/ft})) + (4 \times (-0.60605 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9786 \text{ ft}$$

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

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

$$V_{max} = ((-0.60605 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8142 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9786 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8142 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9786 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.1141 \text{ kip}$$

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

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

$$M_{max} = ((-0.60605 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(8.8142 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9786 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8142 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9786 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (8.8142 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9786 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.237 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.815 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.237 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.237 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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**

<p>25.7.2.2 25.7.2.1</p>	<p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties: Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> <p style="text-align: center;">$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p style="text-align: center;">$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</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>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.815 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0040427$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a)</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters: $b_w = 48 \text{ in}$ - 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> <p style="text-align: center;">$V_{c,max} = 296.21 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.815 \text{ kip} \rightarrow 10815 \text{ lbf}$,</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> <p style="text-align: center;">$V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$</p>	

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10815 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.93 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.93 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.93 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.93 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.03 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.1741 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.1741 \text{ kip})}{(111.03 \text{ kip})}$$

$$\text{Ratio} = 0.073618$$

Status: **PASS**
Ratio: **0.070**

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

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

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

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

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

Allowable flexural strength:

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

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

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

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

14.5.2.1b

 $\phi M_{n,2}$

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

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

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

Therefore,

 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction: $M_{max} = 22.237 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(22.237 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.08909$$

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