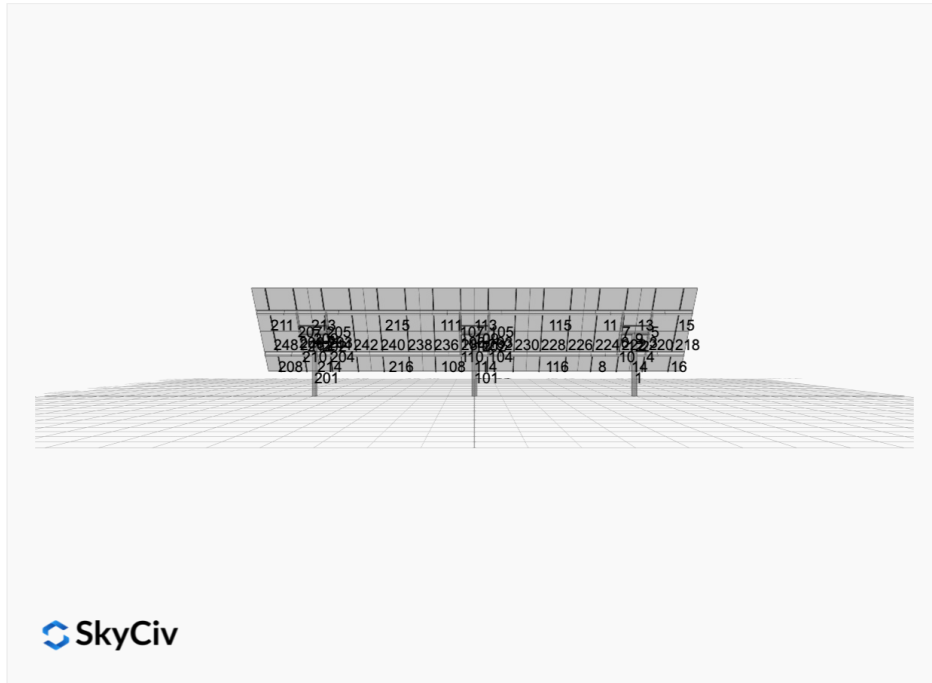


Project Details



Project Name: winthrop 32 mod (580W)
Location: 7831 Farm to Market Rd, Whitefish, MT 59937, USA
Unique ID: 3P-22.5-8TOP-HD-45-L-4Hx8W-2B4D
Dealer: _____

Date: Thu Aug 07 2025
Number of Modules: 32
Number of Poles: 3
Date Sold: _____



Array Dimensions N/S	15.05 ft
Array Dimensions E/W	60.48 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

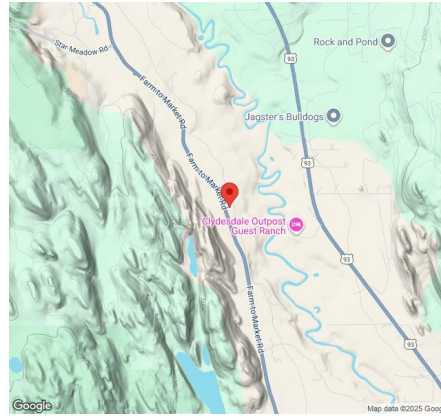
MT Solar Bill of Materials (3P-22.5-8TOP-HD-45-L-4Hx8W-2B4D)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	3
MTS-HF-HD	H-Frame Assembly-HD	3
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	8

Rail Bill of Materials

Part	Qty
Rails (181in)	16
Rail Attachment	32
Module Mid Clamp	48
Module End Clamp	32
Ground Lug	8

Site Details:



Site Address: 7831 Farm to Market Rd, Whitefish, MT 59937, USA

Array Specification

Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	4
Number of Columns:	8
Total Number of Modules:	32
Winter Tilt Angle:	50
Front Edge Clearance:	3
Total Array Height at Tilt:	14.53 ft
Total Frame Length:	60.00 ft
Module Info/Notes:	
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	60.48 ft
Rail Length:	180.60 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	8.76 ft
Number of Poles:	3
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.50 ft Pile 3: 6.00 ft
Foundation Volume:	10.963 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	7831 Farm to Market Rd, Whitefish, MT 59937, USA
Wind Speed:	110 mph
Snow Load:	85 psf

Design Disclaimer

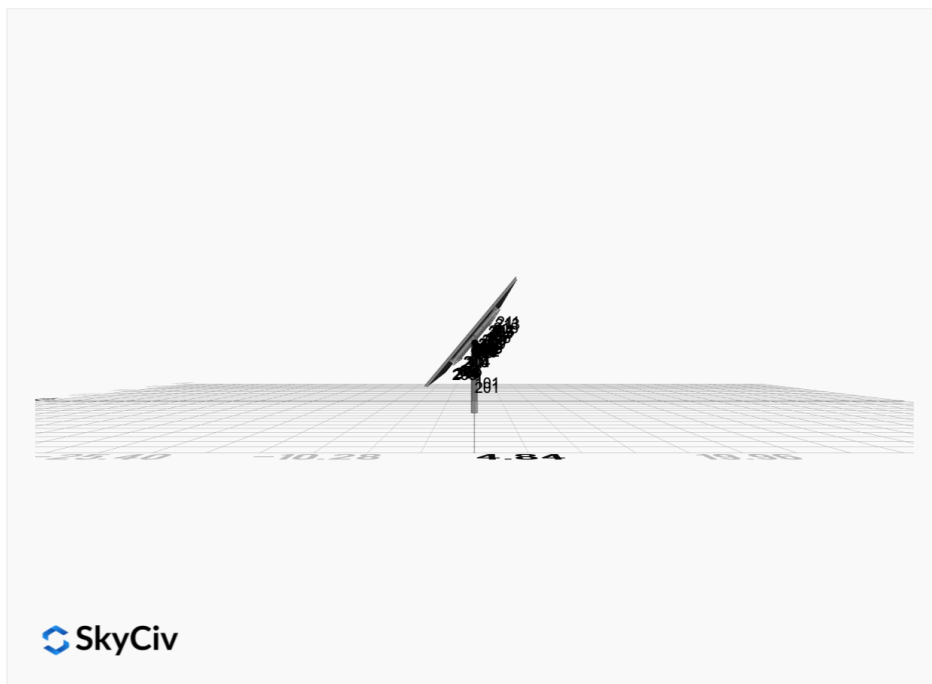
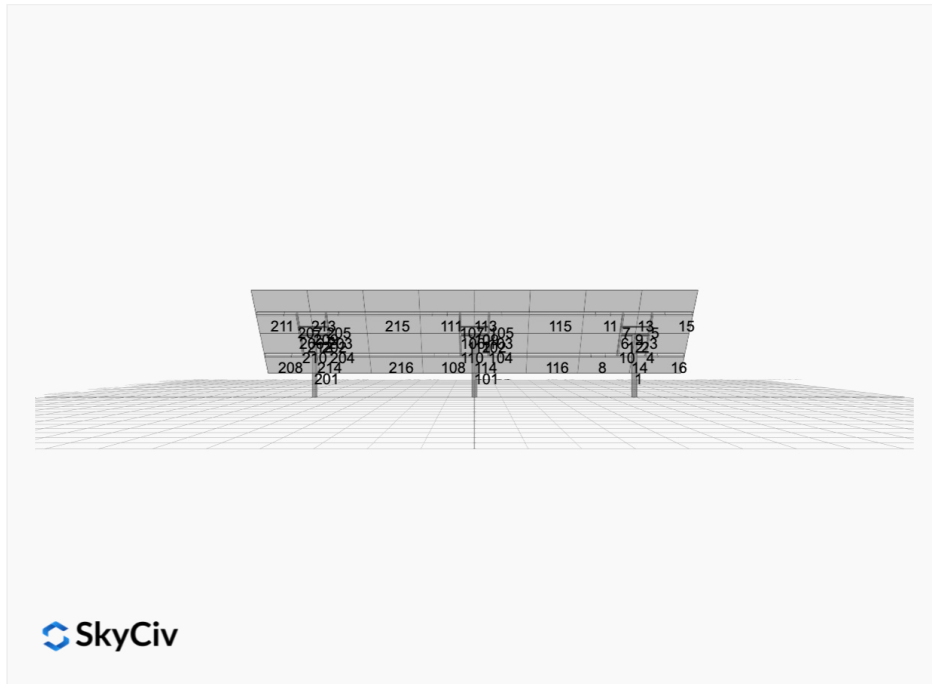
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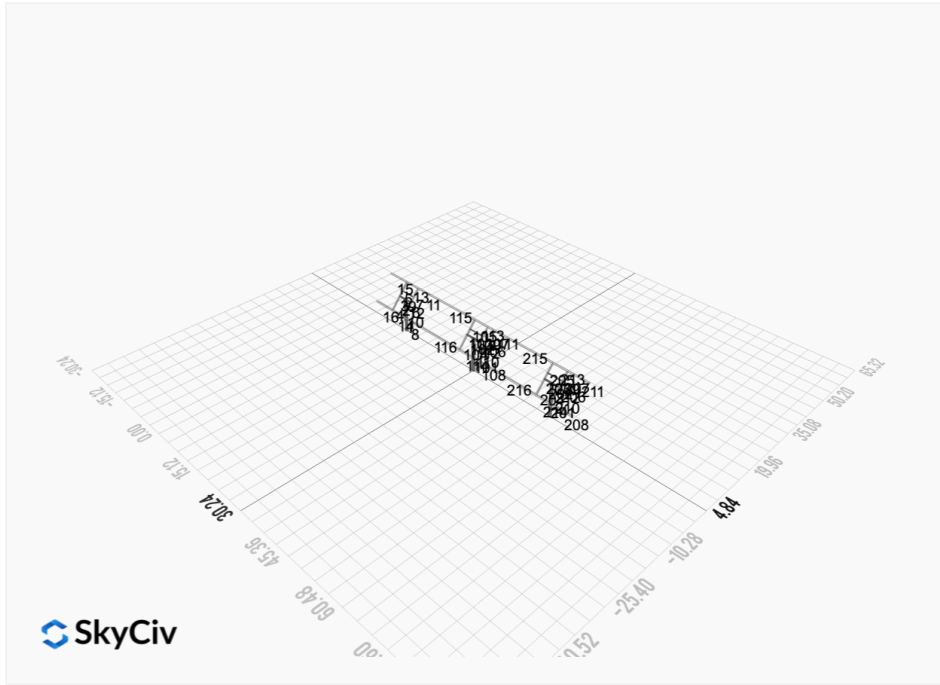
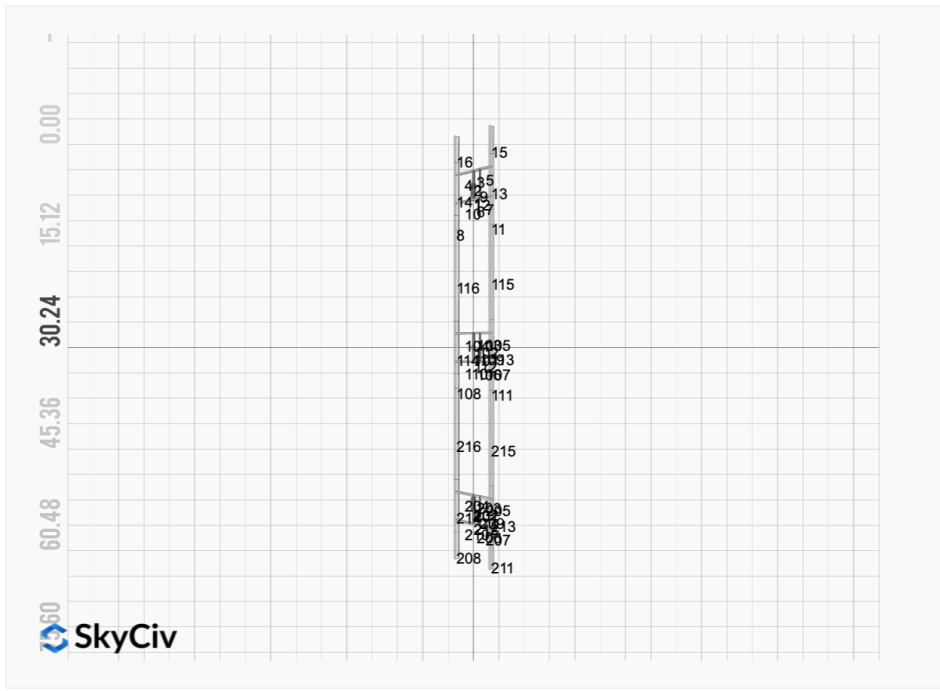
AutoDesigner Input

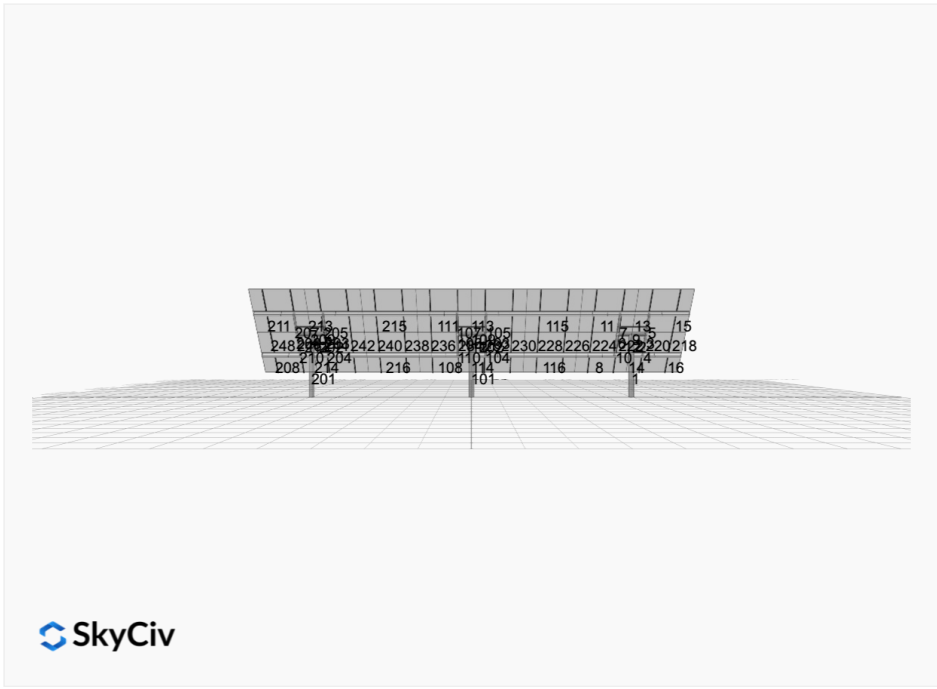
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Design Notes:

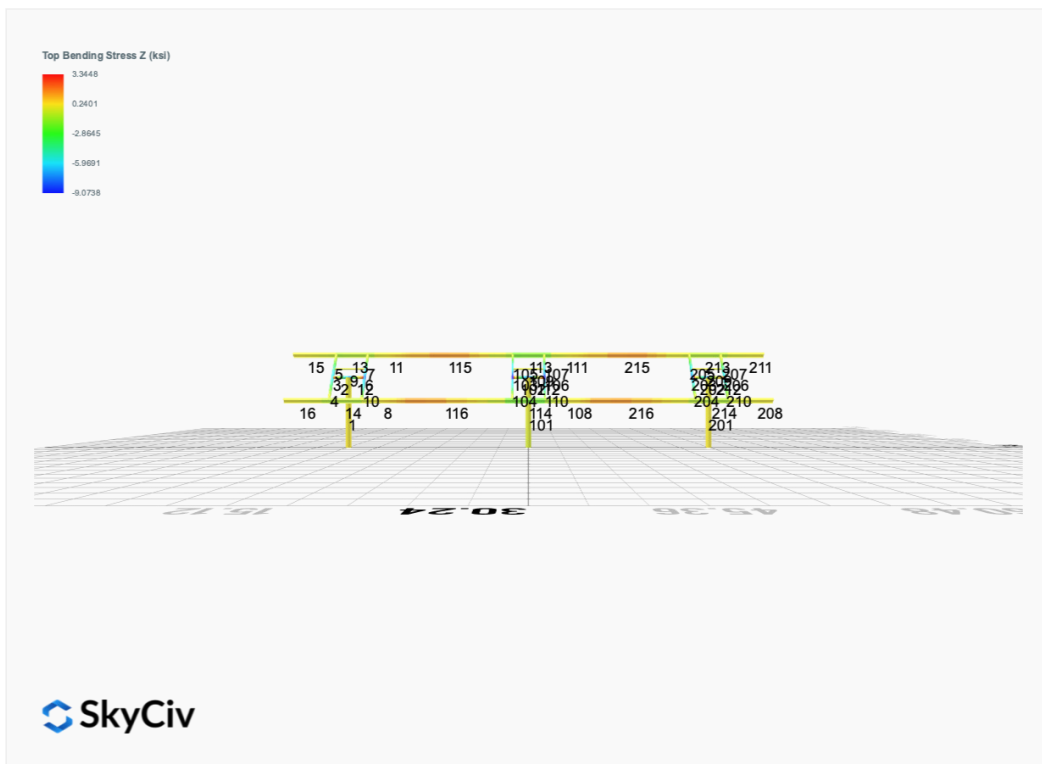
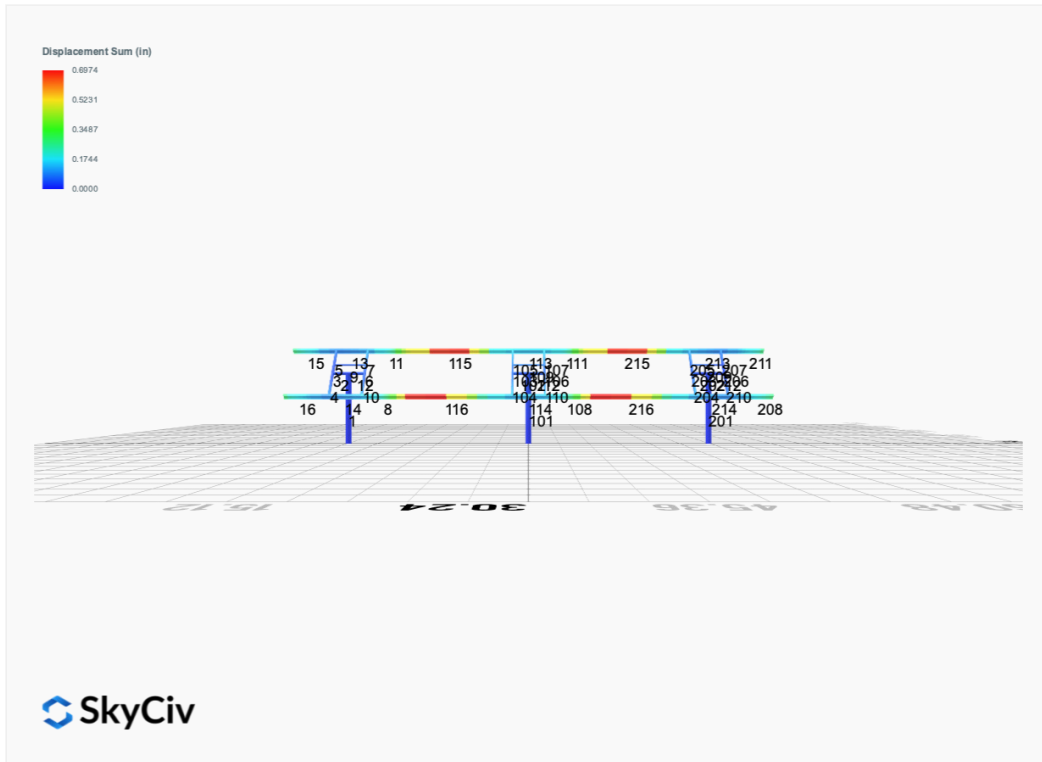
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

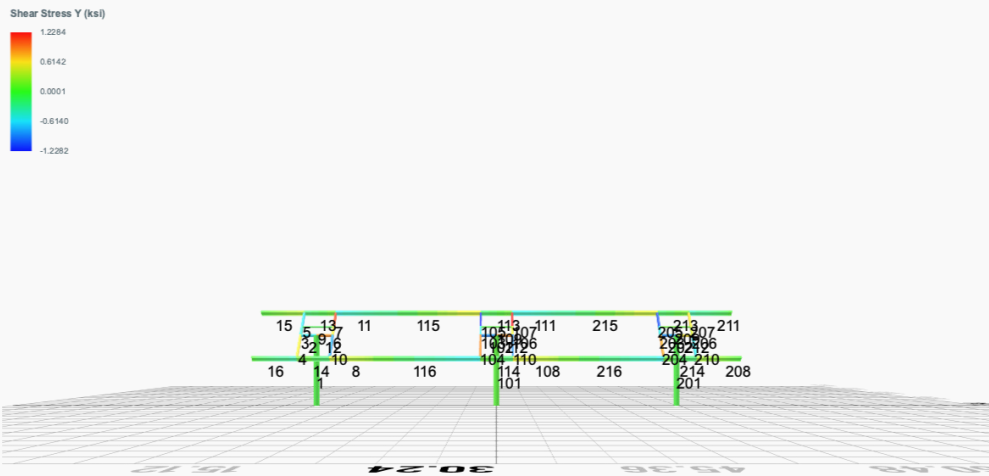
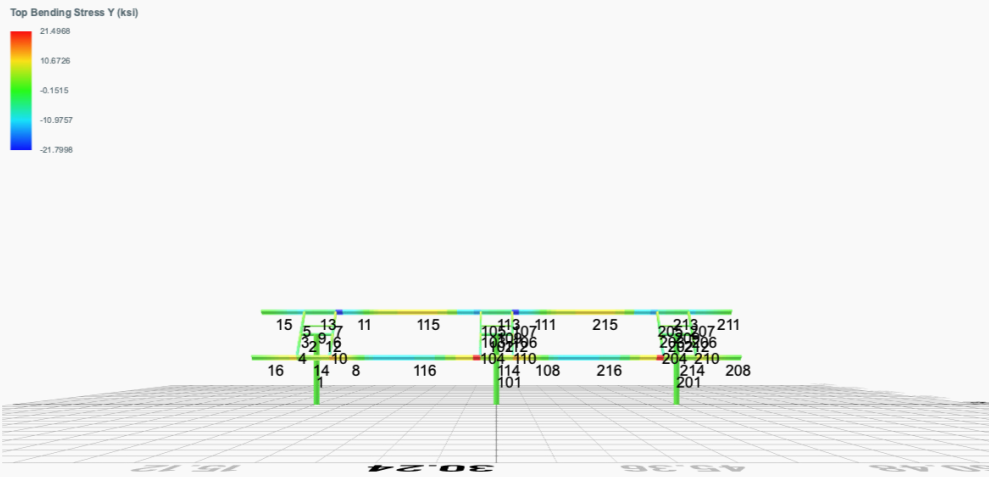


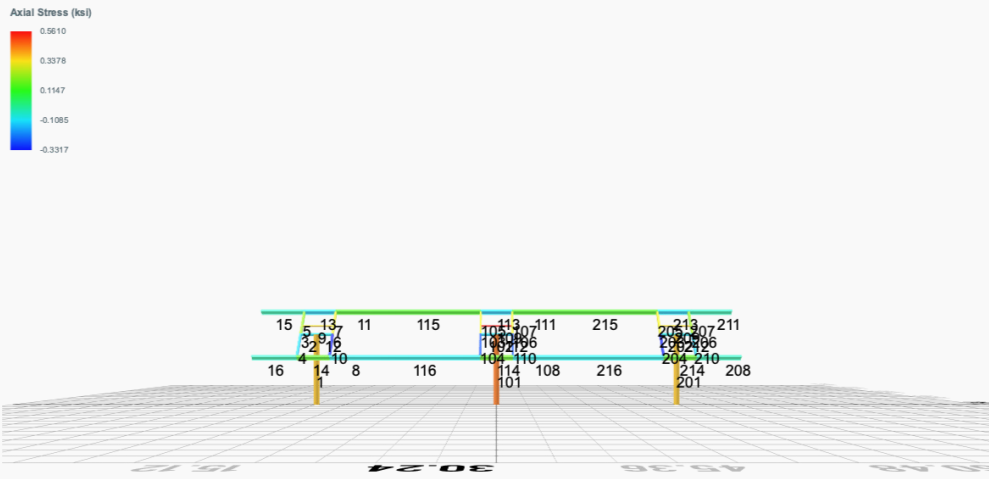




FEM Results (Envelope Worst Case for each member)







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	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 2. D + L	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 3. D + (S or Lr or R)	0.0689	5.4591	0.2176	0.4936	-0.2700	-0.5313
ULS: 3. D + (S or Lr or R)	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0573	4.6214	0.1810	0.4104	-0.2247	-0.4399
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 5b. D + 0.7E	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0573	4.6214	0.1810	0.4104	-0.2247	-0.4399
ULS: 8. 0.6D + 0.7E	0.0136	1.2649	0.0428	0.0966	-0.0532	-0.0993
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8535	4.5100	0.2484	0.5029	-0.9249	25.3666
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8979	-0.2929	-0.1037	-0.1770	0.7396	-25.3265
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0998	6.4228	0.3139	0.6669	-0.8519	18.7092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0573	4.6214	0.1810	0.4104	-0.2247	-0.4399
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2138	2.8205	0.0498	0.1570	0.3965	-19.3106
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0573	4.6214	0.1810	0.4104	-0.2247	-0.4399
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1345	3.9096	0.2042	0.4174	-0.7158	18.9835
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1791	0.3074	-0.0599	-0.0925	0.5326	-19.0362
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0227	2.1082	0.0713	0.1609	-0.0886	-0.1655
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8626	3.6668	0.2199	0.4386	-0.8895	25.4328
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0136	1.2649	0.0428	0.0966	-0.0532	-0.0993
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8889	-1.1362	-0.1322	-0.2414	0.7751	-25.2603
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0136	1.2649	0.0428	0.0966	-0.0532	-0.0993

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	9.8933
Shear X	-4.8431
Shear Z	0.4716
Moment X	1.0298
Moment Y (Twist)	1.5962
Moment Z	42.6755

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	6.4228
Shear X	-2.8979
Shear Z	0.3139
Moment X	0.6669
Moment Y (Twist)	0.9249
Moment Z	25.4328

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	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 2. D + L	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 3. D + (S or Lr or R)	-0.1379	6.6492	-0.0001	-0.0025	-0.0005	1.1800
ULS: 3. D + (S or Lr or R)	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.1148	5.6119	-0.0001	-0.0021	-0.0004	0.9837

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 5b. D + 0.7E	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.1148	5.6119	-0.0001	-0.0021	-0.0004	0.9837
ULS: 8. 0.6D + 0.7E	-0.0272	1.5001	-0.0000	-0.0005	-0.0001	0.2369
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6822	5.5749	-0.0001	-0.0027	-0.0019	32.2685
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5933	-0.5760	0.0001	0.0011	0.0016	-30.9412
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8424	7.9180	-0.0001	-0.0035	-0.0017	24.8890
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.1148	5.6119	-0.0001	-0.0021	-0.0004	0.9837
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6142	3.3048	0.0000	-0.0007	0.0009	-22.5183
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.1148	5.6119	-0.0001	-0.0021	-0.0004	0.9837
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7730	4.8062	-0.0001	-0.0023	-0.0015	24.3001
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6836	0.1930	0.0001	0.0006	0.0012	-23.1072
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0454	2.5001	-0.0000	-0.0008	-0.0002	0.3948
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6641	4.5749	-0.0001	-0.0024	-0.0019	32.1106
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0272	1.5001	-0.0000	-0.0005	-0.0001	0.2369
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6115	-1.5761	0.0001	0.0014	0.0017	-31.0991
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0272	1.5001	-0.0000	-0.0005	-0.0001	0.2369

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	12.1999
Shear X	-6.1604
Shear Z	-0.0002
Moment X	-0.0053
Moment Y (Twist)	0.0032
Moment Z	54.5432

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	7.9180
Shear X	-3.6822
Shear Z	-0.0001
Moment X	-0.0035
Moment Y (Twist)	0.0019
Moment Z	32.2685

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	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 2. D + L	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 3. D + (S or Lr or R)	0.0690	5.4588	-0.2175	-0.4982	0.2674	-0.5325
ULS: 3. D + (S or Lr or R)	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0575	4.6211	-0.1810	-0.4143	0.2225	-0.4409
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 5b. D + 0.7E	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0575	4.6211	-0.1810	-0.4143	0.2225	-0.4409
ULS: 8. 0.6D + 0.7E	0.0136	1.2649	-0.0428	-0.0975	0.0526	-0.0995
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8532	4.5097	-0.2483	-0.5076	0.9195	25.3636
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8977	-0.2928	0.1036	0.1786	-0.7359	-25.3244
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0995	6.4223	-0.3137	-0.6732	0.8463	18.7063
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0575	4.6211	-0.1810	-0.4143	0.2225	-0.4409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2137	2.8204	-0.0498	-0.1585	-0.3952	-19.3098
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0575	4.6211	-0.1810	-0.4143	0.2225	-0.4409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1342	3.9093	-0.2041	-0.4213	0.7116	18.9813
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1790	0.3074	0.0599	0.0933	-0.5300	-19.0348
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0227	2.1081	-0.0713	-0.1624	0.0877	-0.1659
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8623	3.6665	-0.2198	-0.4427	0.8844	25.4300
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0136	1.2649	-0.0428	-0.0975	0.0526	-0.0995
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8886	-1.1360	0.1321	0.2436	-0.7710	-25.2580
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0136	1.2649	-0.0428	-0.0975	0.0526	-0.0995

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	9.8927
Shear X	-4.8428
Shear Z	-0.4714
Moment X	-1.0393
Moment Y (Twist)	1.5868
Moment Z	42.6727

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	6.4223
Shear X	-2.8977
Shear Z	-0.3137
Moment X	-0.6732
Moment Y (Twist)	0.9195
Moment Z	25.4300

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: winthrop 32 mod (580W)
 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				
9	8in Pipe Sch 40	8.63	0.32				

Section Dimensions							

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	

Section Dimensions							

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	377.97	250.02	83.29	83.29	113.39	113.39
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	23.45	6.12	40.24	43.62
114	133.20	85.85	23.52	6.12	40.24	43.62
115	133.20	19.55	12.16	6.12	40.24	43.62
116	133.20	19.55	12.83	6.12	40.24	43.62
201	377.97	250.02	83.29	83.29	113.39	113.39
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	194.54	21.95	21.95	59.50	59.50
213	133.20	85.85	24.69	6.12	40.24	43.62
214	133.20	85.85	24.82	6.12	40.24	43.62
215	133.20	19.55	12.12	6.12	40.24	43.62
216	133.20	19.55	12.26	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	0.040	0.512	0.037	0.043	0.004	0.539	#13	0.376	Not Required	Pass
2	0.006	0.258	0.181	0.064	0.036	0.402	#13	0.054	Not Required	Pass
3	0.010	0.445	0.056	0.043	0.011	0.453	#13	0.045	Not Required	Pass
4	0.009	0.444	0.176	0.045	0.039	0.536	#21	0.080	Not Required	Pass
5	0.009	0.276	0.163	0.044	0.043	0.298	#21	0.074	Not Required	Pass
6	0.015	0.595	0.121	0.061	0.023	0.666	#21	0.045	Not Required	Pass
7	0.016	0.369	0.302	0.059	0.076	0.413	#21	0.074	Not Required	Pass
8	0.004	0.081	0.316	0.040	0.026	0.321	#23	0.095	Not Required	Pass
9	0.021	0.045	0.093	0.004	0.005	0.119	#13	0.204	Not Required	Pass
10	0.017	0.573	0.283	0.057	0.059	0.685	#21	0.080	Not Required	Pass
11	0.007	0.075	0.330	0.042	0.026	0.339	#21	0.095	Not Required	Pass
12	0.004	0.399	0.254	0.097	0.048	0.613	#13	0.053	Not Required	Pass
13	0.009	0.169	0.696	0.052	0.032	0.829	#21	0.286	Not Required	Pass
14	0.010	0.163	0.679	0.050	0.032	0.793	#21	0.190	Not Required	Pass
15	0.000	0.050	0.174	0.022	0.012	0.220	#21	Not Required	Not Required	Pass

15	0.000	0.050	0.174	0.022	0.013	0.220	#21	Not Required	Not Required	Pass
16	0.000	0.050	0.174	0.022	0.013	0.220	#21	Not Required	Not Required	Pass
101	0.049	0.655	0.000	0.054	0.000	0.675	#13	0.376	Not Required	Pass
102	0.007	0.411	0.284	0.100	0.052	0.651	#13	0.035	Not Required	Pass
103	0.015	0.649	0.075	0.065	0.002	0.691	#13	0.045	Not Required	Pass
104	0.015	0.663	0.291	0.066	0.060	0.792	#21	0.080	Not Required	Pass
105	0.015	0.404	0.309	0.064	0.079	0.457	#21	0.074	Not Required	Pass
106	0.015	0.649	0.075	0.065	0.002	0.691	#13	0.045	Not Required	Pass
107	0.015	0.404	0.309	0.064	0.079	0.457	#21	0.074	Not Required	Pass
108	0.004	0.068	0.321	0.047	0.026	0.385	#21	0.095	Not Required	Pass
109	0.031	0.041	0.063	0.001	0.000	0.109	#13	0.204	Not Required	Pass
110	0.015	0.663	0.291	0.066	0.060	0.792	#21	0.080	Not Required	Pass
111	0.007	0.054	0.333	0.045	0.026	0.384	#21	0.095	Not Required	Pass
112	0.007	0.411	0.284	0.100	0.052	0.651	#13	0.035	Not Required	Pass
113	0.009	0.229	0.700	0.055	0.032	0.908	#21	0.286	Not Required	Pass
114	0.012	0.270	0.685	0.057	0.032	0.916	#21	0.286	Not Required	Pass
115	0.042	0.440	0.365	0.045	0.026	0.772	#21	0.925	Not Required	Pass
116	0.008	0.415	0.358	0.047	0.026	0.731	#21	0.925	Not Required	Pass
201	0.040	0.512	0.037	0.043	0.004	0.539	#13	0.376	Not Required	Pass
202	0.004	0.398	0.254	0.097	0.048	0.613	#13	0.053	Not Required	Pass
203	0.015	0.595	0.121	0.061	0.023	0.666	#21	0.045	Not Required	Pass
204	0.017	0.573	0.283	0.057	0.059	0.685	#21	0.080	Not Required	Pass
205	0.016	0.369	0.302	0.059	0.076	0.413	#21	0.074	Not Required	Pass
206	0.010	0.445	0.056	0.043	0.011	0.454	#13	0.045	Not Required	Pass
207	0.009	0.277	0.163	0.044	0.043	0.298	#21	0.074	Not Required	Pass
208	0.000	0.050	0.174	0.022	0.013	0.220	#21	Not Required	Not Required	Pass
209	0.021	0.045	0.093	0.004	0.005	0.118	#13	0.204	Not Required	Pass
210	0.009	0.444	0.176	0.045	0.039	0.536	#21	0.080	Not Required	Pass
211	0.000	0.050	0.174	0.022	0.013	0.220	#21	Not Required	Not Required	Pass
212	0.006	0.258	0.181	0.065	0.036	0.403	#13	0.054	Not Required	Pass
213	0.009	0.169	0.696	0.052	0.032	0.828	#21	0.190	Not Required	Pass
214	0.010	0.163	0.679	0.050	0.032	0.792	#21	0.286	Not Required	Pass
215	0.042	0.449	0.365	0.042	0.026	0.782	#21	0.925	Not Required	Pass
216	0.008	0.435	0.358	0.040	0.026	0.749	#21	0.925	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection

P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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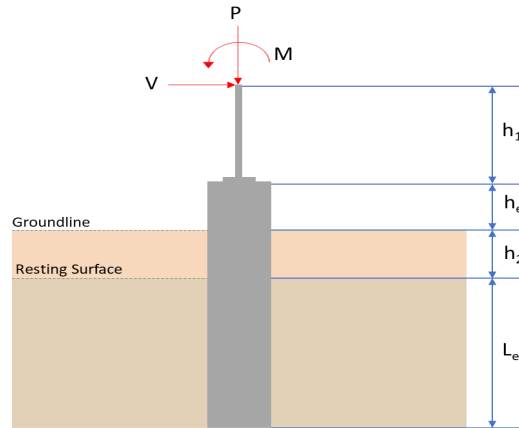
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6$ 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)	6.423	9.893
V_x (kip)	-2.898	-4.843
V_z (kip)	0.314	0.472
M_x (kipft)	0.667	1.030
M_z (kipft)	25.433	42.675

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,

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

$$H_o = \frac{(-2.898 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.46146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.433 \text{ kipft}) + ((-2.898 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0498 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5447 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.314 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.05 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.667 \text{ kipft}) + ((0.314 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10621 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.5233 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5447 \text{ ft}), (2.5233 \text{ ft})]$$

$$L_{e,req} = 5.545 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.545 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.92417$$

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

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

$$q = \frac{(6.423 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40144 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40144 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20072$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46146 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0498 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0498 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.0498 \text{ kipft/ft})) + (4 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1565 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0498 \text{ kipft/ft})) + (3 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (4.0498 \text{ kipft/ft})) + (2 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.1963 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0498 \text{ kipft/ft})) + ((-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.88848 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1565 \text{ ft})}{2}$$

$$p_a = 0.31174 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.1963 \text{ kip/ft}^2)}{(0.31174 \text{ kip/ft}^2)}$$

$$Ratio = 0.62968$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.88848 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.9872$$

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

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

Considering z-direction:

$H_o = 0.05 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10621 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10621 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.05 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.10621 \text{ kipft/ft})) + (4 \times (0.05 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3266 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.10621 \text{ kipft/ft})) + (3 \times (0.05 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.10621 \text{ kipft/ft})) + (2 \times (0.05 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.039806 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10621 \text{ kipft/ft})) + ((0.05 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.085403 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3266 \text{ ft})}{2}$$

$$p_a = 0.32449 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.039806 \text{ kip/ft}^2)}{(0.32449 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.12267$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

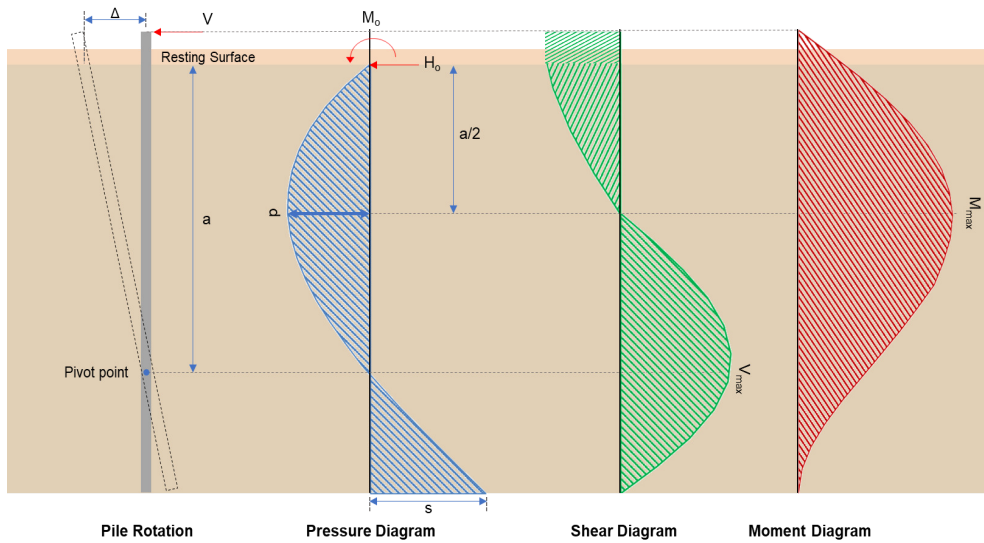
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.085403 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.094893$$

Status: **PASS**
Ratio: **0.120**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.843 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77118 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.675 \text{ kipft}) + ((-4.843 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7954 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7954 \text{ kipft/ft})}{(-0.77118 \text{ kip/ft})}$$

$$E = 8.8117 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7954 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.7954 \text{ kipft/ft})) + (4 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.77118 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.7954 \text{ kipft/ft})) + (4 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1561 \text{ ft}$$

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

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

$$V_{max} = ((-0.77118 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8117 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1561 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8117 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1561 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.05 \text{ kip}$$

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

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

$$M_{max} = ((-0.77118 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(8.8117 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8117 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8117 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.474 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.472 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.075159 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.03 \text{ kipft}) + ((0.472 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16401 \text{ kipft/ft})}{(0.075159 \text{ kip/ft})}$$

$$E = 2.1822 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16401 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.075159 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.16401 \text{ kipft/ft})) + (4 \times (0.075159 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3235 \text{ ft}$$

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

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

$$V_{max} = ((0.075159 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1822 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3235 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.1822 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3235 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39477 \text{ kip}$$

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

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

$$M_{max} = ((0.075159 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.1822 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3235 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.1822 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3235 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1822 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3235 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0241 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.893 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.267 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.267 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(9.893 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0036981$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.893 \text{ kip} \rightarrow 9893 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9893 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.8 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.8 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.8 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.8 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.95 \text{ kip}$$

Considering x-direction:

V_{max} = 10.05 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(10.05 \text{ kip})}{(110.95 \text{ kip})}$$

$$Ratio = 0.09058$$

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

Considering z-direction:

$V_{max} = 0.39477 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.39477 \text{ kip})}{(110.95 \text{ kip})}$$

$$Ratio = 0.003558$$

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:

$M_{max} = 28.474 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(28.474 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11408$$

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

Considering z-direction:

$M_{max} = 1.0241 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.0241 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0041029$$

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

REFERENCES	CALCULATIONS	RESULTS
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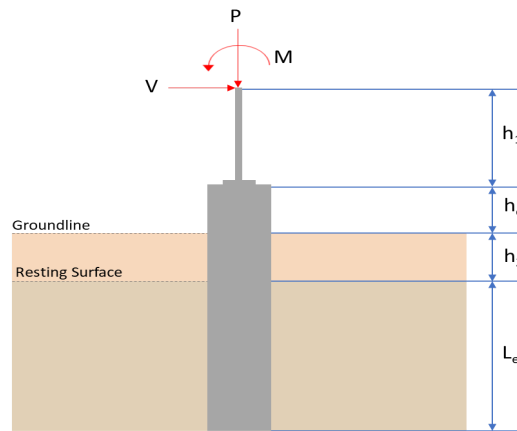
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6$ 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)	6.422	9.893
V_x (kip)	-2.898	-4.843
V_z (kip)	-0.314	-0.471
M_x (kipft)	-0.673	-1.039
M_z (kipft)	25.430	42.673

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,

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

$$H_o = \frac{(-2.898 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.46146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.43 \text{ kipft}) + ((-2.898 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0494 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5444 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.314 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.05 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.673 \text{ kipft}) + ((-0.314 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10717 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5694 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.5444 \text{ ft}), (1.5694 \text{ ft})]$$

$$L_{e,req} = 5.544 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(5.544 \text{ ft})}{(6 \text{ ft})}$$

$$Ratio = 0.924$$

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

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

$$q = \frac{(6.422 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.40138 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.40138 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20069$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.46146 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0494 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0494 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.0494 \text{ kipft/ft})) + (4 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1566 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0494 \text{ kipft/ft})) + (3 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (4.0494 \text{ kipft/ft})) + (2 \times (-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.19624 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0494 \text{ kipft/ft})) + ((-0.46146 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.88832 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1566 \text{ ft})}{2}$$

$$p_a = 0.31174 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19624 \text{ kip/ft}^2)}{(0.31174 \text{ kip/ft}^2)}$$

$$Ratio = 0.62951$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.88832 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.98703$$

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

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

Considering z-direction:

$H_o = -0.05 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10717 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10717 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.10717 \text{ kipft/ft})) + (4 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3256 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.10717 \text{ kipft/ft})) + (3 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.10717 \text{ kipft/ft})) + (2 \times (-0.05 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.016619 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10717 \text{ kipft/ft})) + ((-0.05 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = -0.014278 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3256 \text{ ft})}{2}$$

$$p_a = 0.32442 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.016619 \text{ kip/ft}^2)}{(0.32442 \text{ kip/ft}^2)}$$

$$Ratio = -0.051226$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

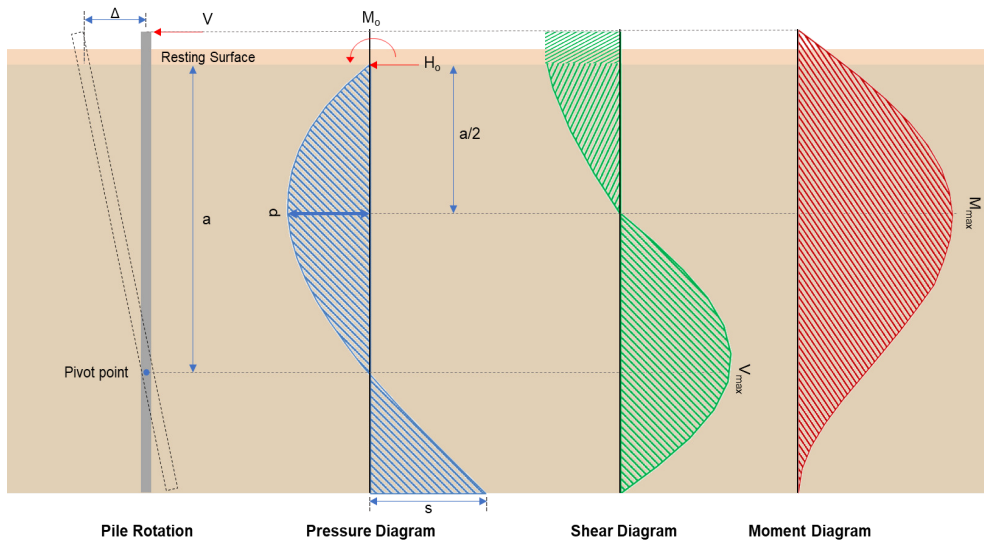
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.014278 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = -0.015865$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.843 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.77118 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.673 \text{ kipft}) + ((-4.843 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7951 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7951 \text{ kipft/ft})}{(-0.77118 \text{ kip/ft})}$$

$$E = 8.8113 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7951 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.7951 \text{ kipft/ft})) + (4 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.77118 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.7951 \text{ kipft/ft})) + (4 \times (-0.77118 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1561 \text{ ft}$$

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

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

$$V_{max} = ((-0.77118 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8113 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1561 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8113 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1561 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.05 \text{ kip}$$

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

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

$$M_{max} = ((-0.77118 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(8.8113 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8113 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8113 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1561 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 28.473 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.471 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.075 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.039 \text{ kipft}) + ((-0.471 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.16545 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.16545 \text{ kipft/ft})}{(-0.075 \text{ kip/ft})}$$

$$E = 2.2059 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.16545 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.075 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.16545 \text{ kipft/ft})) + (4 \times (-0.075 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3223 \text{ ft}$$

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

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

$$V_{max} = ((-0.075 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2059 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3223 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.2059 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3223 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.396 \text{ kip}$$

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

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

$$M_{max} = ((-0.075 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(2.2059 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3223 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.2059 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3223 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2059 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3223 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0281 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.893 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.267 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.267 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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. 10ø: 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;">$Ratio = \frac{(9.893 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0036981$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.893 \text{ kip} \rightarrow 9893 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9893 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.8 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.8 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.8 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.8 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.95 \text{ kip}$$

Considering x-direction:

V_{max} = 10.05 kip - Maximum shear force in the x-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(10.05 \text{ kip})}{(110.95 \text{ kip})}$$

$$Ratio = 0.090577$$

Considering z-direction:

$V_{max} = 0.396 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.396 \text{ kip})}{(110.95 \text{ kip})}$$

$$Ratio = 0.0035691$$

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

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:

$M_{max} = 28.473 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(28.473 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11407$$

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

Considering z-direction:

$M_{max} = 1.0281 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.0281 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.004119$$

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

REFERENCES	CALCULATIONS	RESULTS
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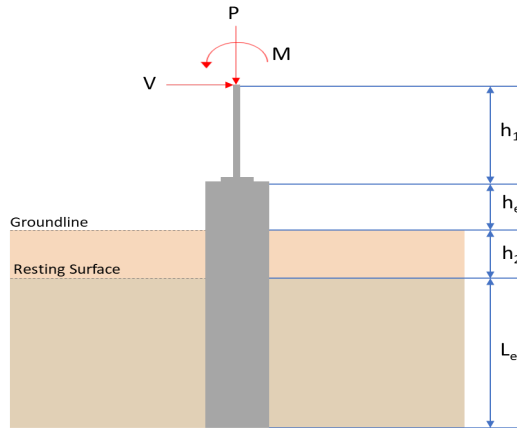
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.5$ 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)	7.918	12.200
V_x (kip)	-3.682	-6.160
V_z (kip)	0.000	0.000
M_x (kipft)	-0.004	-0.005
M_z (kipft)	32.269	54.543

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,

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

$$H_o = \frac{(-3.682 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58631 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(32.269 \text{ kipft}) + ((-3.682 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.1384 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.8866 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.004 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00063694 \text{ kipft/ft}$$

L_e - Required depth of embedment in earth,

$$L_e = 2.29 \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.00063694 \text{ kipft/ft})}{(150 \text{ psf/ft})}}$$

$$L_e = 0.37083 \text{ ft}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8866 \text{ ft}), (0.37083 \text{ ft})]$$

$$L_{e,req} = 5.887 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.887 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.90569$$

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

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

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

$$q = \frac{(7.918 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49487 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49487 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24744$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58631 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.1384 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1384 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.58631 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (5.1384 \text{ kipft/ft})) + (4 \times (-0.58631 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5125 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.1384 \text{ kipft/ft})) + (3 \times (-0.58631 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (5.1384 \text{ kipft/ft})) + (2 \times (-0.58631 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.18948 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.1384 \text{ kipft/ft})) + ((-0.58631 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.91822 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5125 \text{ ft})}{2}$$

$$p_a = 0.33844 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18948 \text{ kip/ft}^2)}{(0.33844 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.55986$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.91822 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94176$$

Status: **PASS**
Ratio: **0.560**

Status: **PASS**
Ratio: **0.940**

Considering z-direction:

$H_o = 0 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.00063694 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00063694 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.00063694 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3333 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.00063694 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.00063694 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.000060302 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.00063694 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.00018091 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3333 \text{ ft})}{2}$$

$$p_a = 0.325 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000060302 \text{ kip/ft}^2)}{(0.325 \text{ kip/ft}^2)}$$

$$Ratio = 0.00018555$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

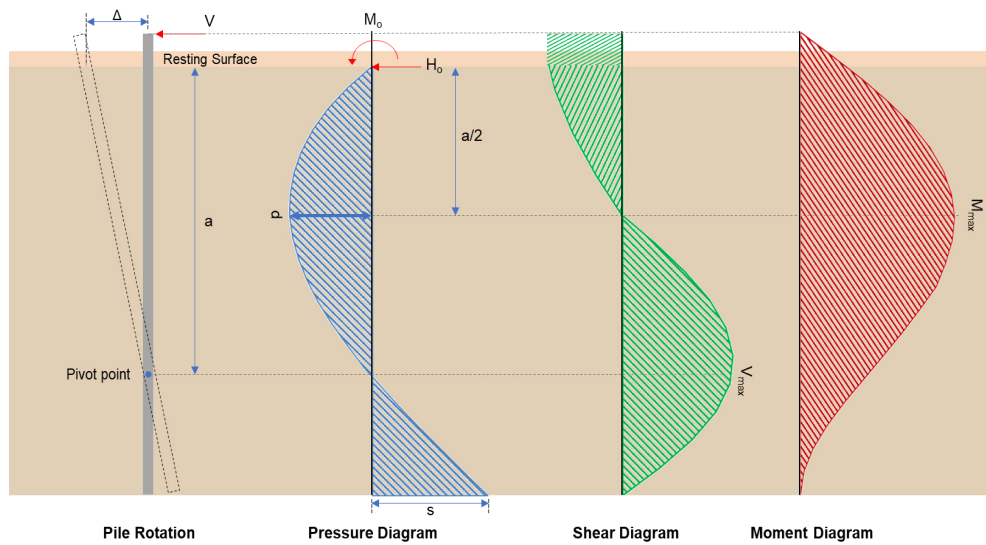
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00018091 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.00018555$$

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,

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

$$H_o = \frac{(-6.16 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.98089 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(54.543 \text{ kipft}) + ((-6.16 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.6852 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.6852 \text{ kipft/ft})}{(-0.98089 \text{ kip/ft})}$$

$$E = 8.8544 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$= (6 M_o) + (4 H_o L_e)$$

$$a = \frac{(4 \times (8.6852 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.98089 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (8.6852 \text{ kipft/ft})) + (4 \times (-0.98089 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5113 \text{ ft}$$

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

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

$$V_{max} = ((-0.98089 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8544 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.5113 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8544 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.5113 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.045 \text{ kip}$$

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

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

$$M_{max} = ((-0.98089 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(8.8544 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.5113 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8544 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.5113 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8544 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.5113 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 36.837 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

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

$$M_o = 0.00079618 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00079618 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.00079618 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.3333 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o b}{L_e} \right) \left(\frac{a}{L_e} - 1 \right) \left(\frac{a}{L_e} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(0.00079618 \text{ kipft/ft}) \times (48 \text{ in})}{(6.5 \text{ ft})} \right) \times \left(\frac{(4.3333 \text{ ft})}{(6.5 \text{ ft})} - 1 \right) \times \left(\frac{(4.3333 \text{ ft})}{(6.5 \text{ ft})} \right)^2$$

$$V_{max} = 0.00087103 \text{ kip}$$

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

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

$$M_{max} = (M_o) \left[1 - \left(\frac{4}{2 L_e} \right) + \left(\frac{3}{2 L_e} \right) \right]$$

$$M_{max} = ((0.00079618 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(4.3333 \text{ ft})^3}{2 \times (6.5 \text{ ft})} \right) + \left(3 \times \frac{(4.3333 \text{ ft})^4}{2 \times (6.5 \text{ ft})} \right) \right]$$

$$M_{max} = 0.0028309 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.2 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.191 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.191 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.2 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0045604$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.2 \text{ kip} \rightarrow 12200 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12200 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.11 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.11 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.11 \text{ kip}$$

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

22.5.1.2 $V_{s,a}$ - 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

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

$$\phi V_n = (0.65) \times ((120.11 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.15 \text{ kip}$$

Considering x-direction:

$V_{max} = 12.045 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.045 \text{ kip})}{(111.15 \text{ kip})}$$

$$\text{Ratio} = 0.10836$$

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

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:

$M_{max} = 36.837 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(36.837 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.14759$$

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

Considering z-direction:

$M_{max} = 0.0028309 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.0028309 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.000011342$$

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