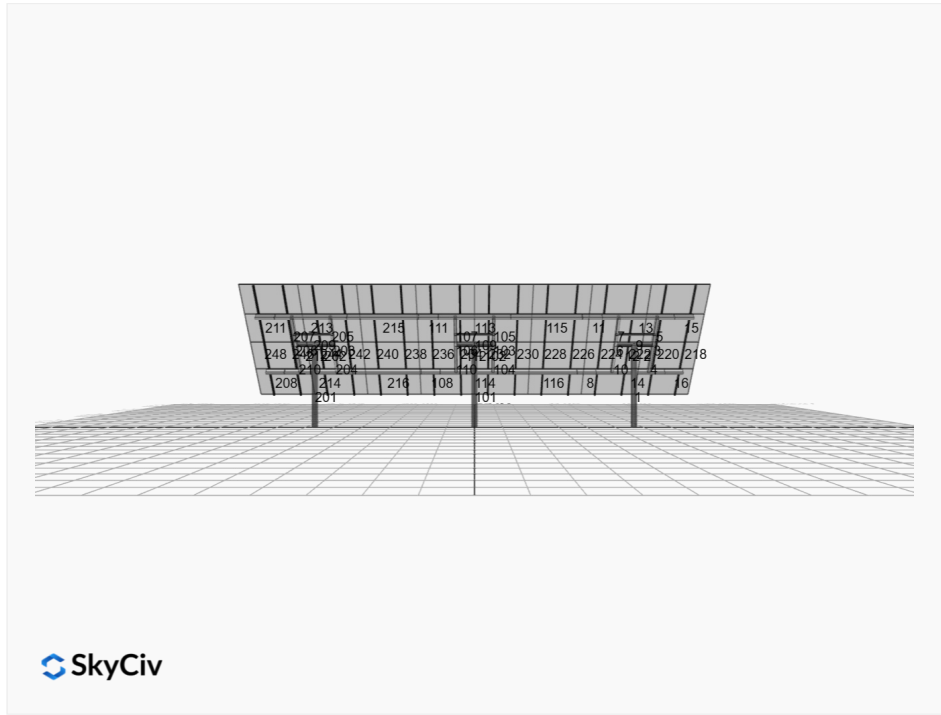


Project Name: MTSOLAR_09080G74824GA **Date:** Tue Aug 19 2025
Location: 6142 Arrowcrest Ct NE, Rockford, MI 49341, USA **Number of Modules:** 32
Unique ID: 3P-17-6TOP-SD-24-L-4Hx8W-D3I2 **Number of Poles:** 3
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	15.03 ft
Array Dimensions E/W	47.93 ft
Winter Tilt Angle	50
Front Edge Clearance	3 ft

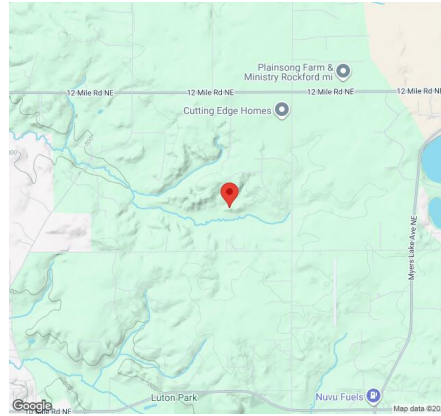
MT Solar Bill of Materials (3P-17-6TOP-SD-24-L-4Hx8W-D3I2)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	3
MTS-HF-SD	H-Frame Assembly-SD	3
MTS-SD-Wing-24	24IN SD Wing	4
MTS-SD-Splice-57	57IN SD Splice	8
MTS-CLAMP-HOOK-4PK	Hook Clamp	8

Rail Bill of Materials

Part	Qty
Rails (180in)	16
Rail Attachment	32
Module Mid Clamp	48
Module End Clamp	32
Ground Lug	8

Site Details:



Site Address: 6142 Arrowcrest Ct NE, Rockford, MI 49341, USA

Array Specification

Duty Classification:	SD
Module Width:	44.60 in
Module Length:	70.90in
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:	14.52 ft
Total Frame Length:	45.50 ft
Module Info/Notes:	
Array Dimensions N/S:	15.03 ft
Array Dimensions E/W:	47.93 ft
Rail Length:	180.40 in
Rail Spacing:	3.00 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	8.76 ft
Number of Poles:	3
Pole Spacing:	17 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 7.00 ft Pile 2: 7.25 ft Pile 3: 7.00 ft
Foundation Volume:	5.563 y ³

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	6142 Arrowcrest Ct NE, Rockford, MI 49341, USA
Wind Speed:	100 mph
Snow Load:	35 psf

Design Disclaimer

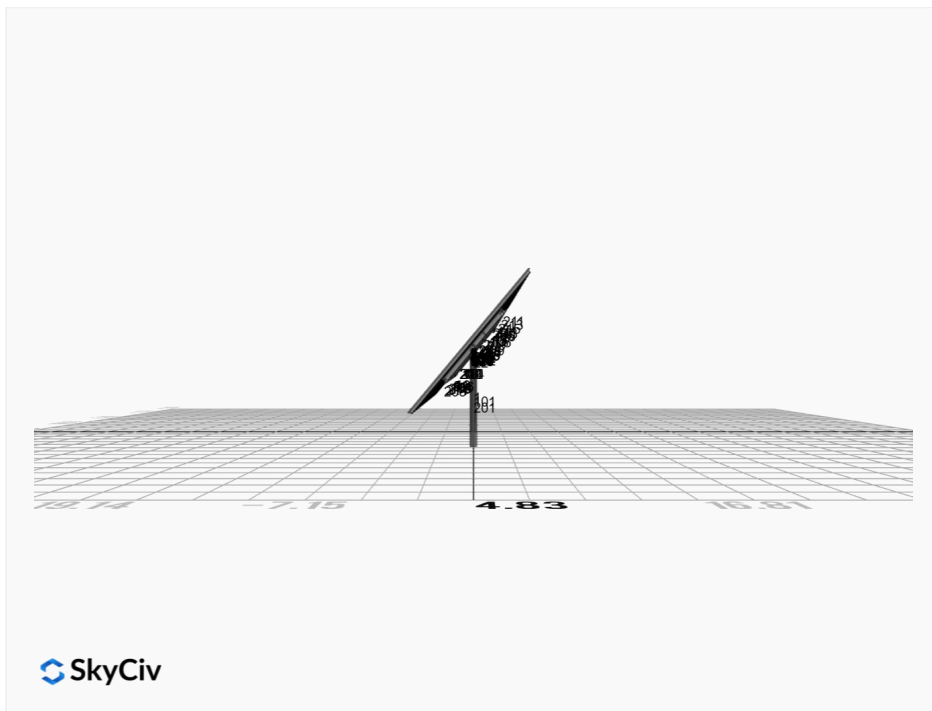
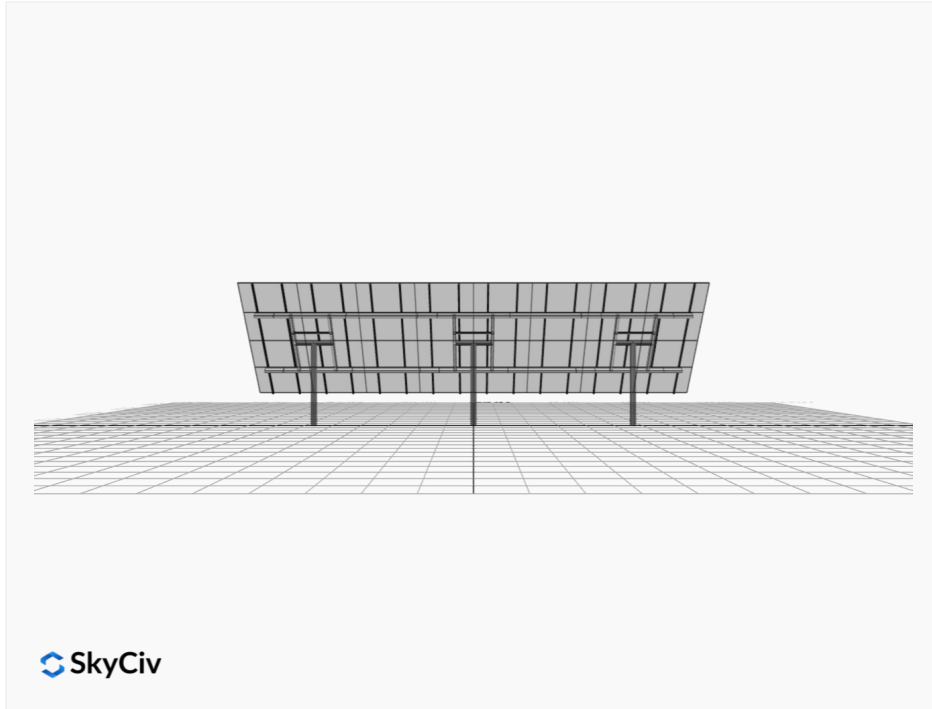
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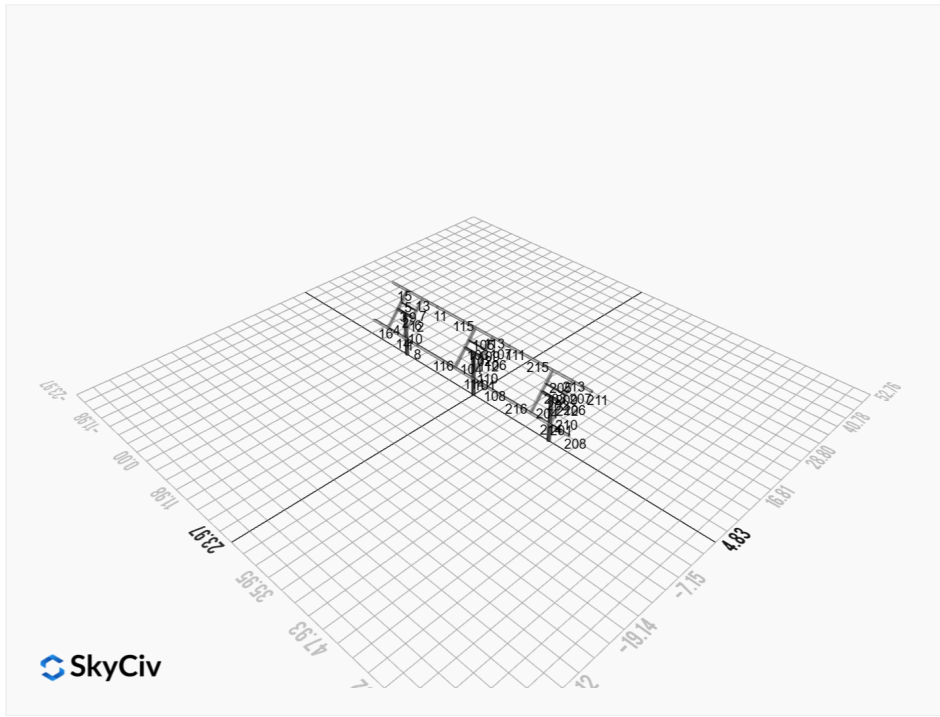
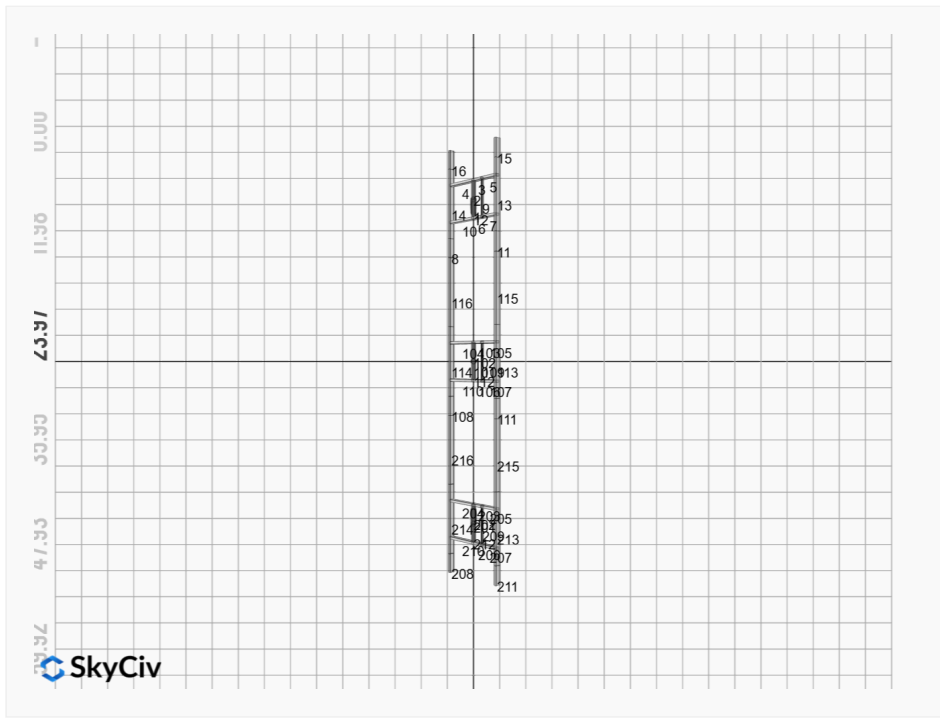
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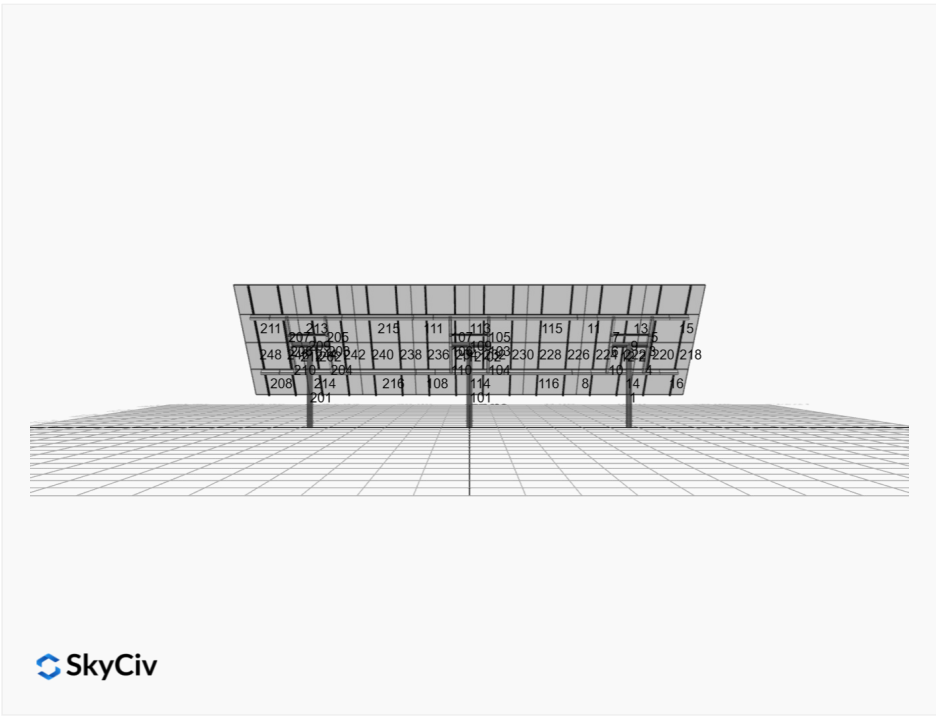
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Design Notes:

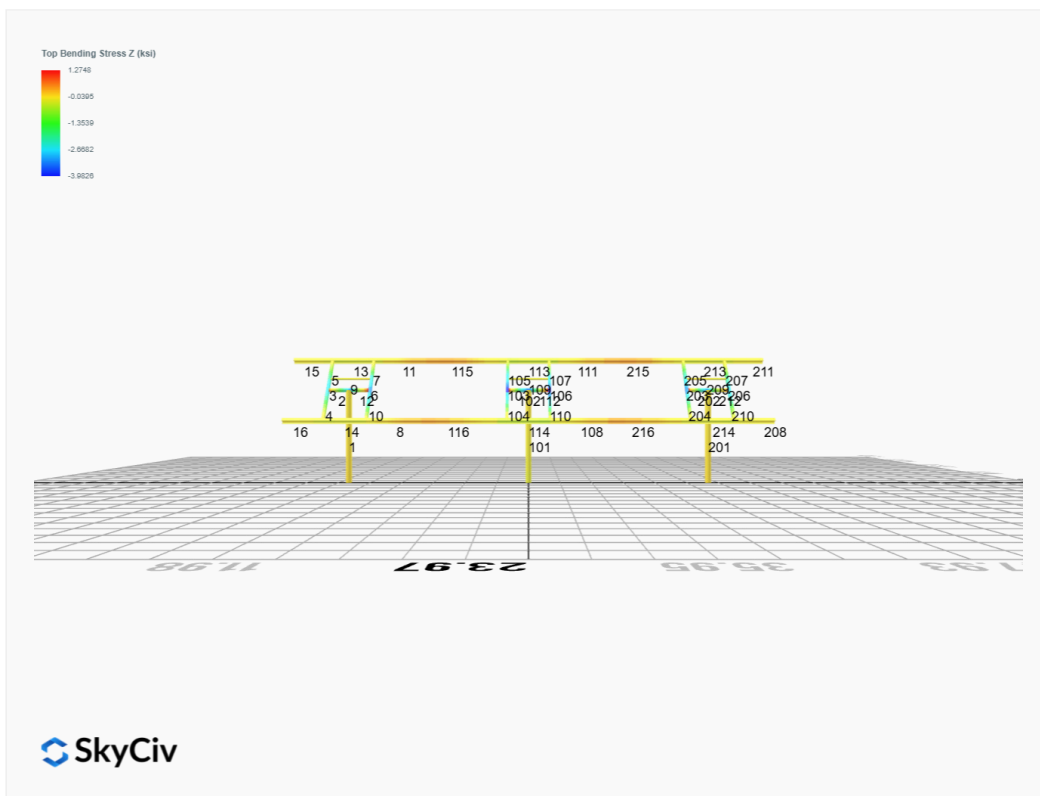
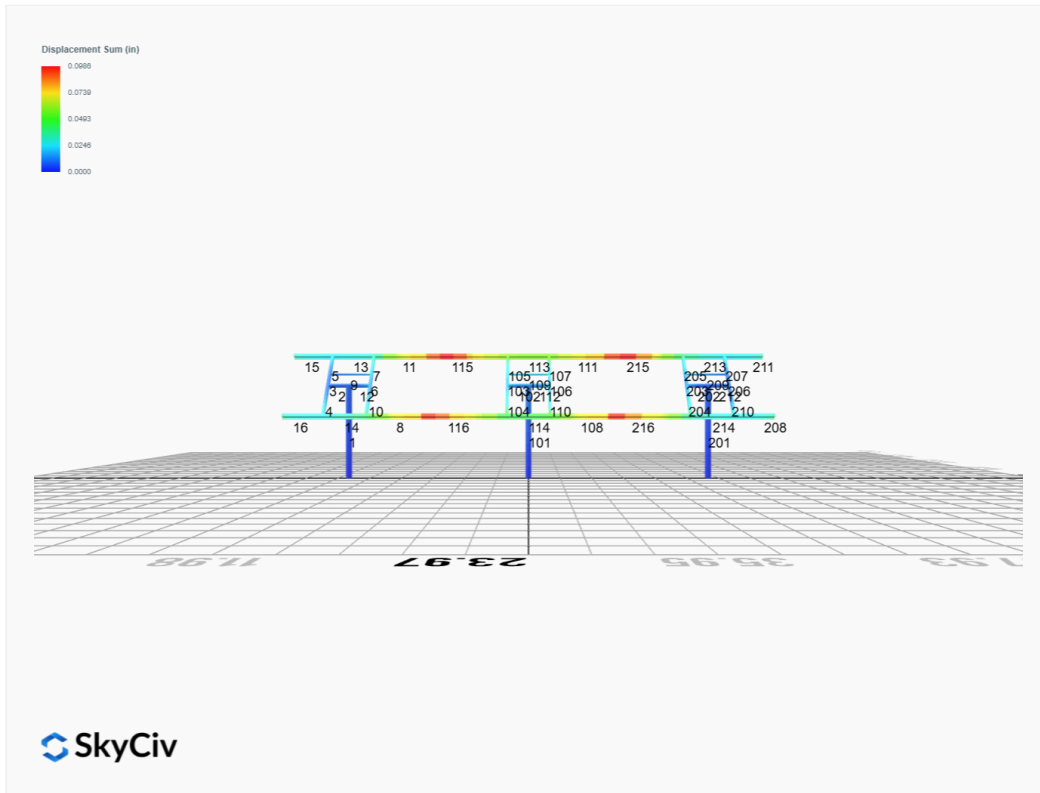
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

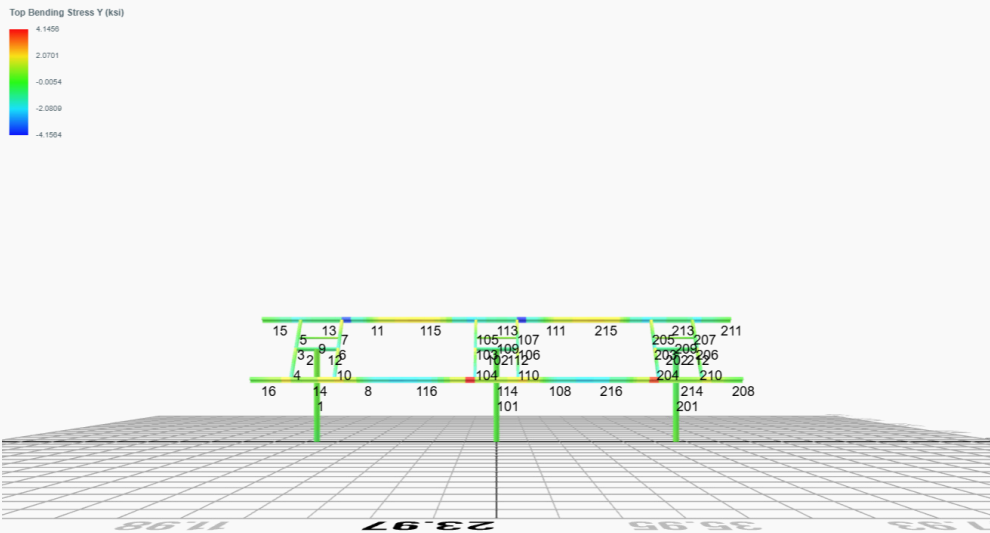




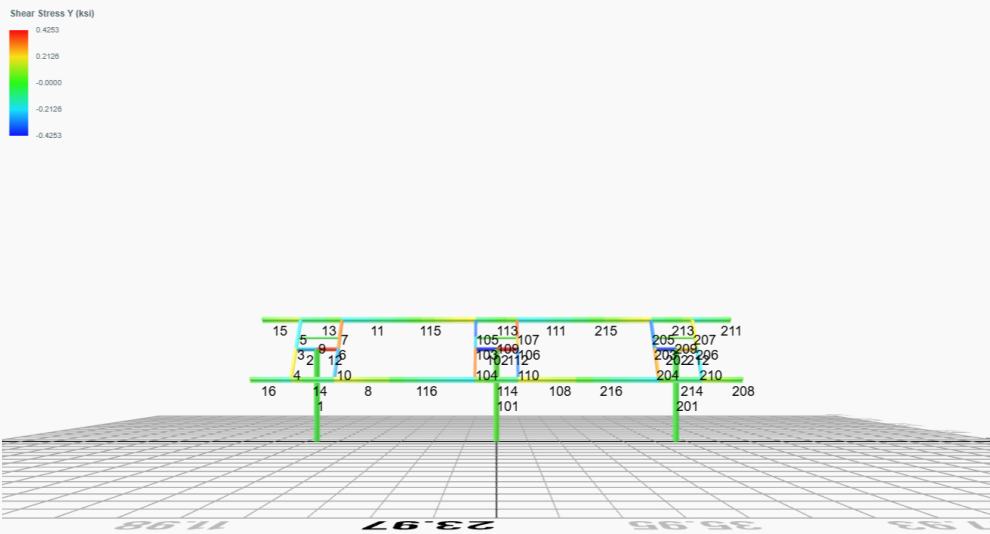


FEM Results (Envelope Worst Case for each member)

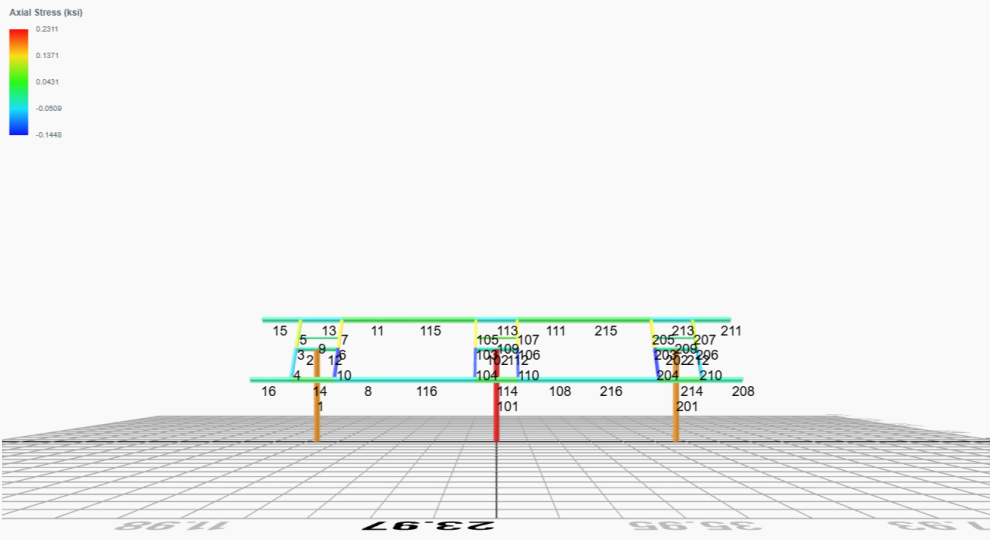




SkyCiv



SkyCiv



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 2. D + L	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 3. D + (S or Lr or R)	0.0245	2.6590	0.0896	0.2324	-0.0829	-0.1655
ULS: 3. D + (S or Lr or R)	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0218	2.3972	0.0795	0.2064	-0.0736	-0.1456
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 5b. D + 0.7E	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0218	2.3972	0.0795	0.2064	-0.0736	-0.1456
ULS: 8. 0.6D + 0.7E	0.0081	0.9671	0.0297	0.0770	-0.0274	-0.0514
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4354	2.8003	0.1394	0.3465	-0.3553	12.8476
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4610	0.4237	-0.0390	-0.0862	0.2593	-12.7706
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0650	3.2886	0.1469	0.3700	-0.3058	9.5543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0218	2.3972	0.0795	0.2064	-0.0736	-0.1456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1074	1.5062	0.0131	0.0454	0.1552	-9.6593
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0218	2.3972	0.0795	0.2064	-0.0736	-0.1456
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0732	2.5032	0.1169	0.2920	-0.2779	9.6143
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0991	0.7207	-0.0169	-0.0326	0.1831	-9.5994
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0135	1.6118	0.0495	0.1284	-0.0457	-0.0856
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4408	2.1556	0.1196	0.2951	-0.3370	12.8818
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0081	0.9671	0.0297	0.0770	-0.0274	-0.0514
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4556	-0.2210	-0.0588	-0.1376	0.2776	-12.7364
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0081	0.9671	0.0297	0.0770	-0.0274	-0.0514

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	4.6004
Shear X	-2.4355
Shear Z	0.2304
Moment X	0.5729
Moment Y (Twist)	0.5932
Moment Z	21.7018

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	3.2886
Shear X	-1.4610
Shear Z	0.1469
Moment X	0.3700
Moment Y (Twist)	0.3553
Moment Z	12.8818

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	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 2. D + L	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 3. D + (S or Lr or R)	-0.0491	3.2023	-0.0002	-0.0013	0.0005	0.3868
ULS: 3. D + (S or Lr or R)	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0436	2.8798	-0.0002	-0.0011	0.0004	0.3449

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 5b. D + 0.7E	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0436	2.8798	-0.0002	-0.0011	0.0004	0.3449
ULS: 8. 0.6D + 0.7E	-0.0163	1.1474	-0.0001	-0.0004	0.0002	0.1316
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.7625	3.4231	-0.0005	-0.0023	0.0006	15.3353
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.7112	0.4006	0.0003	0.0009	-0.0001	-14.5955
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3451	4.0129	-0.0005	-0.0023	0.0007	11.6818
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0436	2.8798	-0.0002	-0.0011	0.0004	0.3449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2602	1.7460	0.0001	0.0000	0.0002	-10.7662
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0436	2.8798	-0.0002	-0.0011	0.0004	0.3449
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.3286	3.0454	-0.0004	-0.0019	0.0006	11.5563
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.2766	0.7785	0.0002	0.0005	0.0000	-10.8918
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0271	1.9123	-0.0001	-0.0007	0.0003	0.2194
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.7516	2.6581	-0.0005	-0.0020	0.0005	15.2475
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0163	1.1474	-0.0001	-0.0004	0.0002	0.1316
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.7221	-0.3643	0.0003	0.0012	-0.0002	-14.6832
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0163	1.1474	-0.0001	-0.0004	0.0002	0.1316

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	5.6173
Shear X	-2.9327
Shear Z	-0.0009
Moment X	-0.0039
Moment Y (Twist)	0.0010
Moment Z	25.8857

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	4.0129
Shear X	-1.7625
Shear Z	-0.0005
Moment X	-0.0023
Moment Y (Twist)	0.0007
Moment Z	15.3353

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	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 2. D + L	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 3. D + (S or Lr or R)	0.0246	2.6589	-0.0893	-0.2333	0.0810	-0.1663
ULS: 3. D + (S or Lr or R)	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0218	2.3971	-0.0793	-0.2072	0.0719	-0.1462
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 5b. D + 0.7E	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0218	2.3971	-0.0793	-0.2072	0.0719	-0.1462
ULS: 8. 0.6D + 0.7E	0.0081	0.9670	-0.0296	-0.0773	0.0268	-0.0516
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.4352	2.8001	-0.1388	-0.3474	0.3516	12.8460
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.4609	0.4238	0.0388	0.0861	-0.2577	-12.7698
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860

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	-1.0648	3.2884	-0.1464	-0.3710	0.3021	9.5528
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0218	2.3971	-0.0793	-0.2072	0.0719	-0.1462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1073	1.5061	-0.0132	-0.0459	-0.1549	-9.6591
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0218	2.3971	-0.0793	-0.2072	0.0719	-0.1462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0730	2.5030	-0.1165	-0.2928	0.2749	9.6130
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.0991	0.7208	0.0167	0.0323	-0.1821	-9.5989
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0136	1.6117	-0.0494	-0.1289	0.0447	-0.0860
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.4407	2.1554	-0.1191	-0.2958	0.3337	12.8804
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0081	0.9670	-0.0296	-0.0773	0.0268	-0.0516
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.4555	-0.2209	0.0585	0.1377	-0.2756	-12.7354
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0081	0.9670	-0.0296	-0.0773	0.0268	-0.0516

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	4.6001
Shear X	-2.4354
Shear Z	-0.2295
Moment X	-0.5743
Moment Y (Twist)	0.5870
Moment Z	21.6995

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	3.2884
Shear X	-1.4609
Shear Z	-0.1464
Moment X	-0.3710
Moment Y (Twist)	0.3516
Moment Z	12.8804

Project Details

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

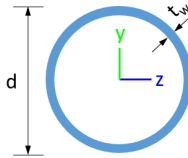


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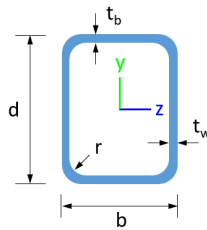
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Design Materials			
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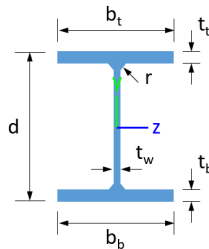
Section Dimensions



ID	Name	d (in)	t _w (in)				
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4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12	



ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I _{yp} (in ⁴)	I _{zp} (in ⁴)	I _w (in ⁶)	S _{yp} (in ³)	S _{zp} (in ³)
----	------	----------------------	----------------------	------------------------------------	------------------------------------	-----------------------------------	------------------------------------	------------------------------------

101	251.10	123.93	42.30	42.30	75.35	75.35
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.89	10.99	6.26	29.14	16.61
104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.03	18.20	6.45	30.09	45.74
114	120.60	84.03	18.49	6.45	30.09	45.74
115	120.60	84.26	18.61	6.45	30.09	45.74
116	120.60	59.18	20.04	6.45	30.09	45.74
201	251.16	123.93	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	10.99	6.26	29.14	16.61
208	120.60	96.18	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.84	10.99	6.26	29.14	16.61
211	120.60	96.18	23.36	6.45	30.09	45.74
212	142.83	140.22	16.17	16.17	42.85	42.85
213	120.60	84.03	20.50	6.45	30.09	45.74
214	120.60	84.03	20.72	6.45	30.09	45.74
215	120.60	84.26	19.58	6.45	30.09	45.74
216	120.60	59.18	19.31	6.45	30.09	45.74

Design Ratio

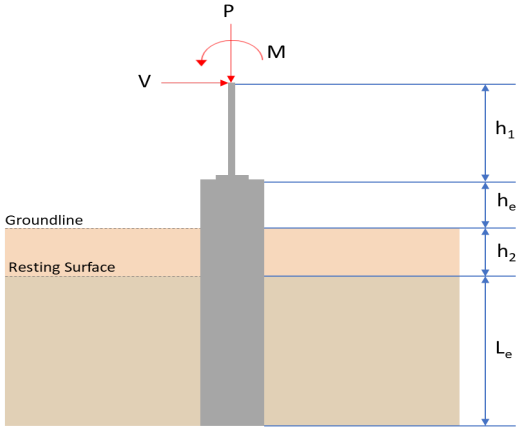
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.037	0.513	0.034	0.032	0.003	0.545	#13	0.491	Not Required	Pass
2	0.001	0.170	0.130	0.043	0.026	0.296	#13	0.079	Not Required	Pass
3	0.007	0.343	0.048	0.034	0.010	0.376	#13	0.044	Not Required	Pass
4	0.006	0.340	0.077	0.034	0.015	0.406	#13	0.078	Not Required	Pass
5	0.007	0.212	0.058	0.034	0.010	0.216	#13	0.073	Not Required	Pass
6	0.010	0.440	0.109	0.045	0.025	0.517	#13	0.044	Not Required	Pass
7	0.011	0.273	0.139	0.044	0.027	0.293	#13	0.073	Not Required	Pass
8	0.002	0.071	0.064	0.023	0.009	0.121	#21	0.088	Not Required	Pass
9	0.001	0.030	0.049	0.002	0.002	0.075	#13	0.198	Not Required	Pass
10	0.010	0.417	0.135	0.042	0.024	0.452	#13	0.078	Not Required	Pass
11	0.002	0.070	0.062	0.025	0.009	0.120	#21	0.088	Not Required	Pass
12	0.001	0.255	0.167	0.060	0.033	0.423	#13	0.052	Not Required	Pass
13	0.003	0.070	0.186	0.034	0.013	0.214	#21	0.265	Not Required	Pass
14	0.003	0.065	0.185	0.032	0.013	0.201	#21	0.177	Not Required	Pass
15	0.000	0.014	0.027	0.011	0.004	0.038	#21	Not Required	Not Required	Pass

16	0.000	0.014	0.027	0.011	0.004	0.038	#21	Not Required	Not Required	Pass
101	0.045	0.612	0.000	0.039	0.000	0.634	#13	0.491	Not Required	Pass
102	0.001	0.261	0.184	0.063	0.034	0.446	#13	0.052	Not Required	Pass
103	0.011	0.468	0.088	0.047	0.018	0.527	#13	0.044	Not Required	Pass
104	0.010	0.487	0.138	0.049	0.024	0.564	#13	0.078	Not Required	Pass
105	0.010	0.290	0.142	0.047	0.027	0.314	#13	0.073	Not Required	Pass
106	0.011	0.467	0.088	0.047	0.018	0.527	#13	0.044	Not Required	Pass
107	0.010	0.290	0.142	0.047	0.027	0.314	#13	0.073	Not Required	Pass
108	0.002	0.056	0.060	0.027	0.009	0.097	#21	0.088	Not Required	Pass
109	0.004	0.026	0.038	0.001	0.000	0.066	#13	0.198	Not Required	Pass
110	0.010	0.487	0.138	0.049	0.025	0.564	#13	0.078	Not Required	Pass
111	0.002	0.070	0.060	0.025	0.009	0.106	#21	0.088	Not Required	Pass
112	0.001	0.261	0.184	0.063	0.034	0.445	#13	0.052	Not Required	Pass
113	0.003	0.071	0.194	0.034	0.013	0.251	#21	0.265	Not Required	Pass
114	0.004	0.104	0.194	0.036	0.013	0.271	#21	0.265	Not Required	Pass
115	0.003	0.113	0.107	0.025	0.009	0.197	#21	0.321	Not Required	Pass
116	0.002	0.101	0.108	0.027	0.009	0.190	#21	0.493	Not Required	Pass
201	0.037	0.513	0.034	0.032	0.003	0.544	#13	0.491	Not Required	Pass
202	0.001	0.255	0.167	0.060	0.033	0.422	#13	0.052	Not Required	Pass
203	0.010	0.440	0.109	0.045	0.025	0.517	#13	0.044	Not Required	Pass
204	0.010	0.417	0.135	0.042	0.024	0.452	#13	0.078	Not Required	Pass
205	0.011	0.273	0.138	0.044	0.027	0.293	#13	0.073	Not Required	Pass
206	0.007	0.344	0.048	0.034	0.010	0.377	#13	0.044	Not Required	Pass
207	0.007	0.213	0.058	0.034	0.010	0.216	#13	0.073	Not Required	Pass
208	0.000	0.014	0.027	0.011	0.004	0.038	#21	Not Required	Not Required	Pass
209	0.001	0.030	0.049	0.002	0.002	0.075	#13	0.198	Not Required	Pass
210	0.006	0.341	0.077	0.034	0.015	0.406	#13	0.078	Not Required	Pass
211	0.000	0.014	0.027	0.011	0.004	0.038	#21	Not Required	Not Required	Pass
212	0.001	0.170	0.130	0.043	0.026	0.297	#13	0.079	Not Required	Pass
213	0.003	0.070	0.186	0.034	0.013	0.213	#21	0.177	Not Required	Pass
214	0.003	0.065	0.185	0.032	0.013	0.200	#21	0.265	Not Required	Pass
215	0.003	0.112	0.106	0.025	0.009	0.197	#21	0.321	Not Required	Pass
216	0.002	0.107	0.108	0.023	0.009	0.194	#21	0.493	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: round $D = 36$ in - Pile diameter $L = 7$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1061 1227 1162"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="655 1267 940 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>3.289</td> <td>4.600</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.461</td> <td>-2.436</td> </tr> <tr> <td>V_z (kip)</td> <td>0.147</td> <td>0.230</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.370</td> <td>0.573</td> </tr> <tr> <td>M_z (kipft)</td> <td>12.882</td> <td>21.702</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	3.289	4.600	V_x (kip)	-1.461	-2.436	V_z (kip)	0.147	0.230	M_x (kipft)	0.370	0.573	M_z (kipft)	12.882	21.702	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-1.461 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.487 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p>																											

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

$$M_o = \frac{(12.882 \text{ kipft}) + ((-1.461 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.294 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.3021 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.147 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.049 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.37 \text{ kipft}) + ((0.147 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.12333 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.1011 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.3021 \text{ ft}), (3.1011 \text{ ft})]$$

$$L_{e,req} = 6.302 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.302 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.90029$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(3.289 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.4653 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4653 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23265$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.487 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.294 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.294 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.294 \text{ kipft/ft})) + (4 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8686 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (4.294 \text{ kipft/ft})) + (3 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (4.294 \text{ kipft/ft})) + (2 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.19144 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (4.294 \text{ kipft/ft})) + ((-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.99616 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8686 \text{ ft})}{2}$$

$$p_a = 0.36514 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19144 \text{ kip/ft}^2)}{(0.36514 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.52429$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.99616 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94873$$

Status: **PASS**
Ratio: **0.520**

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

Considering z-direction:

$H_o = 0.049 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.12333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.049 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.12333 \text{ kipft/ft})) + (4 \times (0.049 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0456 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.12333 \text{ kipft/ft})) + (3 \times (0.049 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.12333 \text{ kipft/ft})) + (2 \times (0.049 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.05276 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.12333 \text{ kipft/ft})) + ((0.049 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.11342 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0456 \text{ ft})}{2}$$

$$p_a = 0.37842 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.05276 \text{ kip/ft}^2)}{(0.37842 \text{ kip/ft}^2)}$$

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

$$\text{Ratio} = 0.13942$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

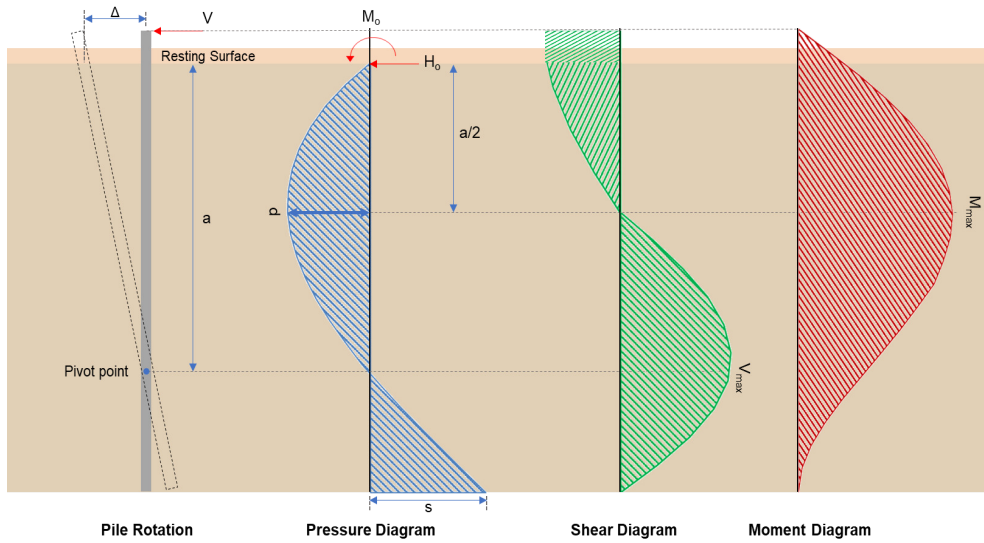
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.11342 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.10802$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.436 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.812 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.702 \text{ kipft}) + ((-2.436 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.234 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.234 \text{ kipft/ft})}{(-0.812 \text{ kip/ft})}$$

$$E = 8.9089 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.234 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.812 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (7.234 \text{ kipft/ft})) + (4 \times (-0.812 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8672 \text{ ft}$$

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

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

$$V_{max} = ((-0.812 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.9089 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8672 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.9089 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8672 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0926 \text{ kip}$$

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

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

$$M_{max} = ((-0.812 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(8.9089 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8672 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.9089 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8672 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.9089 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8672 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.282 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.23 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.076667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.573 \text{ kipft}) + ((0.23 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.191 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.191 \text{ kipft/ft})}{(0.076667 \text{ kip/ft})}$$

$$E = 2.4913 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.191 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (0.076667 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.191 \text{ kipft/ft})) + (4 \times (0.076667 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.047 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right] / \left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((0.076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4913 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.047 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.4913 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.047 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.2989 \text{ kip}$$

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

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

$$M_{max} = ((0.076667 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(2.4913 \text{ ft})}{(7 \text{ ft})} + \frac{(5.047 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.4913 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.047 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4913 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.047 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.90315 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(4.6 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.23 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.23 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(4.6 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0036685$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 4.6 \text{ kip} \rightarrow 4600 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(4600 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.219 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.219 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.219 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.219 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.703 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.0926 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.0926 \text{ kip})}{(73.703 \text{ kip})}$$

$$Ratio = 0.096232$$

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

Considering z-direction:

$V_{max} = 0.2989 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.2989 \text{ kip})}{(73.703 \text{ kip})}$$

$$Ratio = 0.0040554$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 23.282 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.282 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.37536$$

Status: **PASS**
Ratio: **0.380**

Considering z-direction:

$M_{max} = 0.90315 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.90315 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.014561$$

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

REFERENCES	CALCULATIONS	RESULTS
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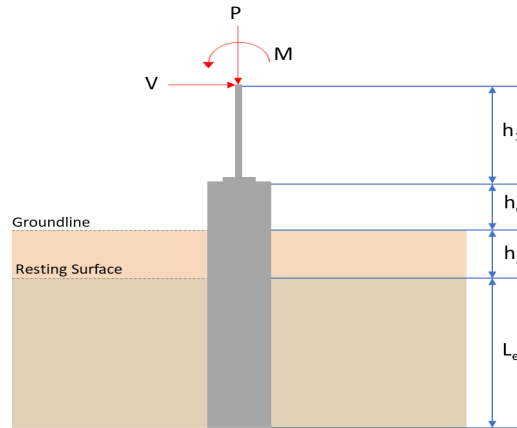
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 7$ 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)	3.288	4.600
V_x (kip)	-1.461	-2.435
V_z (kip)	-0.146	-0.230
M_x (kipft)	-0.371	-0.574
M_z (kipft)	12.880	21.700

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,

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

$$H_o = \frac{(-1.461 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.487 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(12.88 \text{ kipft}) + ((-1.461 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.2933 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.3016 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.146 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.048667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.371 \text{ kipft}) + ((-0.146 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.12367 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8977 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.3016 \text{ ft}), (1.8977 \text{ ft})]$$

$$L_{e,req} = 6.302 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.302 \text{ ft})}{(7 \text{ ft})}$$

$$\text{Ratio} = 0.90029$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(3.288 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.46516 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46516 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23258$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.487 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.2933 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2933 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.2933 \text{ kipft/ft})) + (4 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8686 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (4.2933 \text{ kipft/ft})) + (3 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (4.2933 \text{ kipft/ft})) + (2 \times (-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.19136 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (4.2933 \text{ kipft/ft})) + ((-0.487 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.99591 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8686 \text{ ft})}{2}$$

$$p_a = 0.36514 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19136 \text{ kip/ft}^2)}{(0.36514 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.52406$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.99591 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94848$$

Status: **PASS**
Ratio: **0.520**

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

Considering z-direction:

$H_o = -0.048667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.12367 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12367 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.048667 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.12367 \text{ kipft/ft})) + (4 \times (-0.048667 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0443 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.12367 \text{ kipft/ft})) + (3 \times (-0.048667 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (0.12367 \text{ kipft/ft})) + (2 \times (-0.048667 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = -0.021542 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.12367 \text{ kipft/ft})) + ((-0.048667 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = -0.017952 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0443 \text{ ft})}{2}$$

$$p_a = 0.37833 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.021542 \text{ kip/ft}^2)}{(0.37833 \text{ kip/ft}^2)}$$

$$(0.01000 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.056941$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

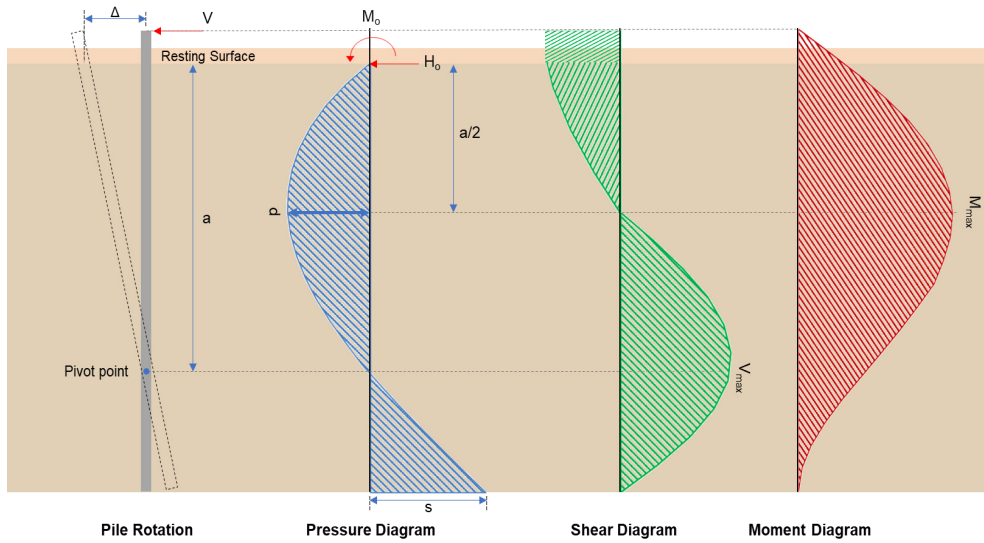
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.017952 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.017098$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.435 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.81167 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.7 \text{ kipft}) + ((-2.435 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 7.2333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2333 \text{ kipft/ft})}{(-0.81167 \text{ kip/ft})}$$

$$E = 8.9117 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2333 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.81167 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (7.2333 \text{ kipft/ft})) + (4 \times (-0.81167 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.8671 \text{ ft}$$

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

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

$$V_{max} = ((-0.81167 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.9117 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8671 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.9117 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8671 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0914 \text{ kip}$$

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

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

$$M_{max} = ((-0.81167 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(8.9117 \text{ ft})}{(7 \text{ ft})} + \frac{(4.8671 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.9117 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.8671 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.9117 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.8671 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.279 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.23 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.076667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.574 \text{ kipft}) + ((-0.23 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.19133 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19133 \text{ kipft/ft})}{(-0.076667 \text{ kip/ft})}$$

$$E = 2.4957 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19133 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.076667 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (0.19133 \text{ kipft/ft})) + (4 \times (-0.076667 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 5.0467 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right] / \left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.076667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.4957 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0467 \text{ ft})}{(7 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.4957 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0467 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.29914 \text{ kip}$$

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

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

$$M_{max} = ((-0.076667 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(2.4957 \text{ ft})}{(7 \text{ ft})} + \frac{(5.0467 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.4957 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(5.0467 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.4957 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(5.0467 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.90402 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(4.6 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.23 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.23 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{1.8322 \text{ in}^2}{1.8408 \text{ in}^2}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(4.6 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0036685$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 4.6 \text{ kip} \rightarrow 4600 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(4600 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.219 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.219 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.219 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.219 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.703 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.0914 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.0914 \text{ kip})}{(73.703 \text{ kip})}$$

$$Ratio = 0.096216$$

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

Considering z-direction:

$V_{max} = 0.29914 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.29914 \text{ kip})}{(73.703 \text{ kip})}$$

$$Ratio = 0.0040588$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 23.279 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.279 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.3753$$

Status: **PASS**
Ratio: **0.380**

Considering z-direction:

$M_{max} = 0.90402 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.90402 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.014575$$

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

REFERENCES	CALCULATIONS	RESULTS
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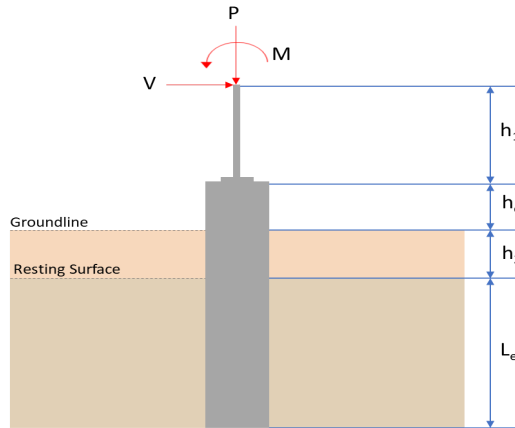
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 7.25$ 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)	4.013	5.617
V_x (kip)	-1.762	-2.933
V_z (kip)	-0.001	-0.001
M_x (kipft)	-0.002	-0.004
M_z (kipft)	15.335	25.886

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,

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

$$H_o = \frac{(-1.762 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.58733 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(15.335 \text{ kipft}) + ((-1.762 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.1117 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.543 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.001 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.00033333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.002 \text{ kipft}) + ((-0.001 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.00066667 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.41466 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.543 \text{ ft}), (0.41466 \text{ ft})]$$

$$L_{e,req} = 6.543 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.543 \text{ ft})}{(7.25 \text{ ft})}$$

$$\text{Ratio} = 0.90248$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.013 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.56772 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.56772 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28386$$

Status: **PASS**
Ratio: **0.280**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.25 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.4167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58733 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.1117 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.1117 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.58733 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (5.1117 \text{ kipft/ft})) + (4 \times (-0.58733 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0491 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.1117 \text{ kipft/ft})) + (3 \times (-0.58733 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (5.1117 \text{ kipft/ft})) + (2 \times (-0.58733 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = 0.19347 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.1117 \text{ kipft/ft})) + ((-0.58733 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = 1.0696 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.0491 \text{ ft})}{2}$$

$$p_a = 0.37868 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19347 \text{ kip/ft}^2)}{(0.37868 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.5109$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0696 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98356$$

Status: **PASS**
Ratio: **0.510**

Status: **PASS**
Ratio: **0.980**

Considering z-direction:

$H_o = -0.00033333 \text{ kip/ft}$ - Lateral force per length of pile.

$M_o = 0.00066667 \text{ kipft/ft}$ - Overturning moment per length of pile.

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00066667 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.00066667 \text{ kipft/ft})) + (4 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.2607 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.00066667 \text{ kipft/ft})) + (3 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft}))]^2}{(7.25 \text{ ft})^2 \times [(3 \times (0.00066667 \text{ kipft/ft})) + (2 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft}))]}$$

$$p = -0.00016616 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.00066667 \text{ kipft/ft})) + ((-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft}))]}{(7.25 \text{ ft})^2}$$

$$s = -0.00019425 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.2607 \text{ ft})}{2}$$

$$p_a = 0.39455 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00016616 \text{ kip/ft}^2)}{(0.39455 \text{ kip/ft}^2)}$$

(0.00200 kip/ft)

$$Ratio = -0.00042115$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.25 \text{ ft})$$

$$p_s = 1.0875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

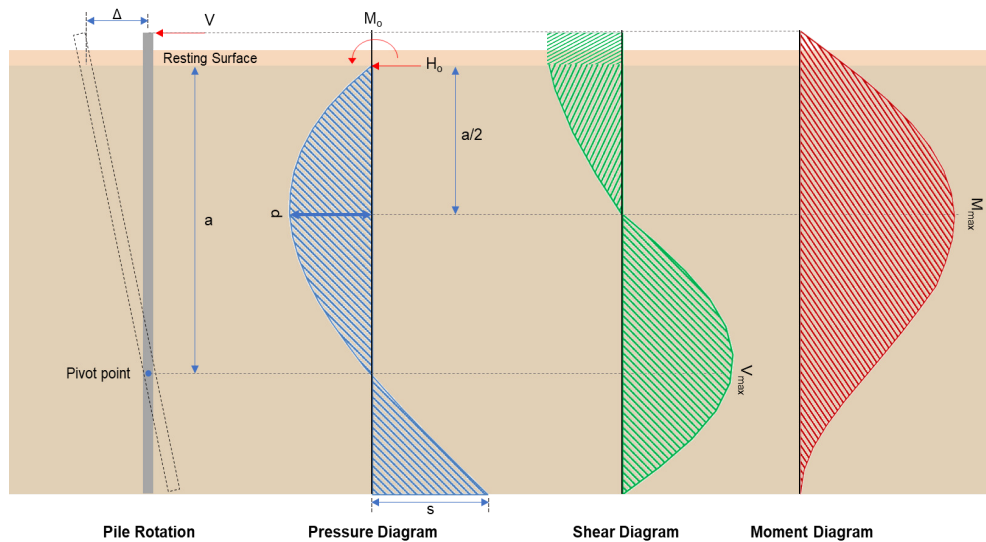
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00019425 \text{ kip/ft}^2)}{(1.0875 \text{ kip/ft}^2)}$$

$$Ratio = -0.00017862$$

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,

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

$$H_o = \frac{(-2.933 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.97767 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(25.886 \text{ kipft}) + ((-2.933 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 8.6287 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(8.6287 \text{ kipft/ft})}{(-0.97767 \text{ kip/ft})}$$

$$E = 8.8258 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.6287 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.97767 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (8.6287 \text{ kipft/ft})) + (4 \times (-0.97767 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.0471 \text{ ft}$$

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

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

$$V_{max} = ((-0.97767 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.8258 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0471 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.8258 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0471 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2527 \text{ kip}$$

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

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

$$M_{max} = ((-0.97767 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(8.8258 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.0471 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.8258 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.0471 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.8258 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.0471 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.995 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.001 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.00033333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.004 \text{ kipft}) + ((-0.001 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.0013333 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0013333 \text{ kipft/ft})}{(-0.00033333 \text{ kip/ft})}$$

$$E = 4 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0013333 \text{ kipft/ft}) \times (7.25 \text{ ft})) + (3 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft})^2)}{(6 \times (0.0013333 \text{ kipft/ft})) + (4 \times (-0.00033333 \text{ kip/ft}) \times (7.25 \text{ ft}))}$$

$$a = 5.1639 \text{ ft}$$

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

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

$$[\setminus L_e \quad / \setminus L_e /]]$$

$$V_{max} = ((-0.00033333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1639 \text{ ft})}{(7.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1639 \text{ ft})}{(7.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.0016416 \text{ kip}$$

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

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

$$M_{max} = ((-0.00033333 \text{ kip/ft}) \times (36 \text{ in}) \times (7.25 \text{ ft})) \times \left[\left(\frac{(4 \text{ ft})}{(7.25 \text{ ft})} + \frac{(5.1639 \text{ ft})}{2 \times (7.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (4 \text{ ft})}{(7.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.1639 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4 \text{ ft})}{(7.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.1639 \text{ ft})}{2 \times (7.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.0053031 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(5.617 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -37.198 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-37.198 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(5.617 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0044796$</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>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 5.617 \text{ kip} \rightarrow 5617 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(5617 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 75.391 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (75.391 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 75.391 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((75.391 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 73.815 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.2527 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.2527 \text{ kip})}{(73.815 \text{ kip})}$$

$$Ratio = 0.1118$$

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

Considering z-direction:

$V_{max} = 0.0016416 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.0016416 \text{ kip})}{(73.815 \text{ kip})}$$

$$Ratio = 0.00002239$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 27.995 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(27.995 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.45133$$

Status: **PASS**
Ratio: **0.450**

Considering z-direction:

$M_{max} = 0.0053031 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{\quad}{\phi M_n}$$

$$Ratio = \frac{(0.0053031 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.000085497$$

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