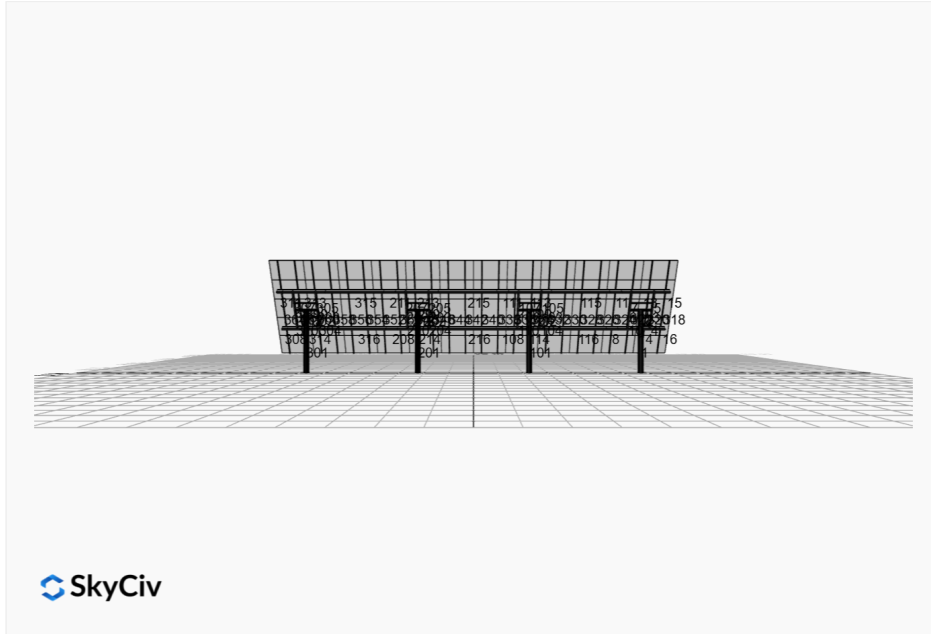


Project Name: Sajani Inampudi 60 Modules
Location: 6125 Prominence Pointe Dr, Anchorage, AK 99516, USA
Unique ID: 4P-19.75-10TOP-XD-12-L-5Hx12W-ILJD
Dealer: _____

Date: Thu Jan 30 2025
Number of Modules: 60
Number of Poles: 4
Date Sold: _____



Array Dimensions N/S	18.81 ft
Array Dimensions E/W	70.37 ft
Winter Tilt Angle	60
Front Edge Clearance	3 ft

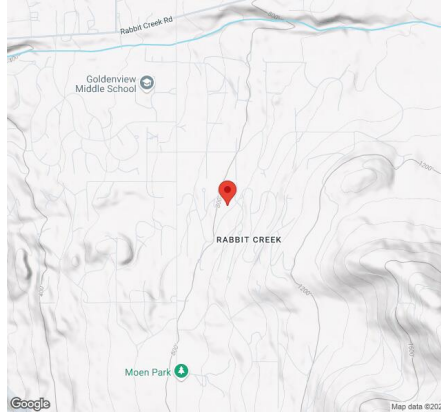
MT Solar Bill of Materials (4P-19.75-10TOP-XD-12-L-5Hx12W-ILJD)

Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

Rail Bill of Materials

Part	Qty
Rails (223in)	24
Rail Attachment	96
Module Mid Clamp	96
Module End Clamp	48
Ground Lug	12

Site Details:



Site Address: 6125 Prominence Pointe Dr, Anchorage, AK 99516, USA

Array Specification

Duty Classification:	XD
Module Width:	44.65 in
Module Length:	69.37in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Winter Tilt Angle:	60
Front Edge Clearance:	3
Total Array Height at Tilt:	19.29 ft
Total Frame Length:	68.75 ft
Frame Weight:	6881 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	70.37 ft
Rail Length:	225.75 in
Rail Spacing:	2.93 ft

Support Specifications

Pole Size:	10in Pipe Sch 80
Pole Length above Grade:	11.15 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 8.00 ft Pile 2: 8.75 ft Pile 3: 8.75 ft Pile 4: 8.00 ft
Foundation Volume:	19.852 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	6125 Prominence Pointe Dr, Anchorage, AK 99516, USA
Wind Speed:	124 mph

Snow Load:

104 psf

Design Disclaimer

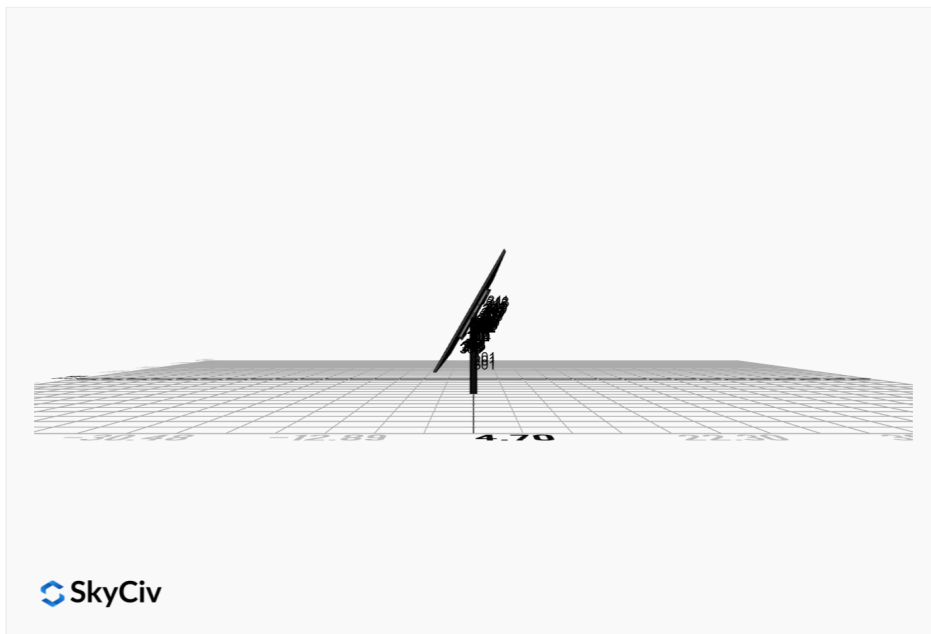
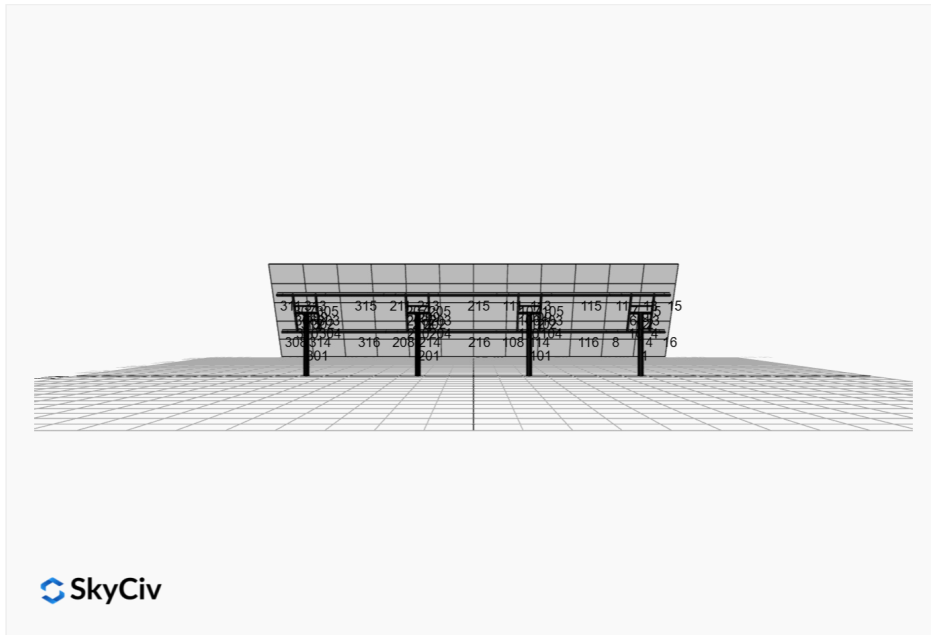
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

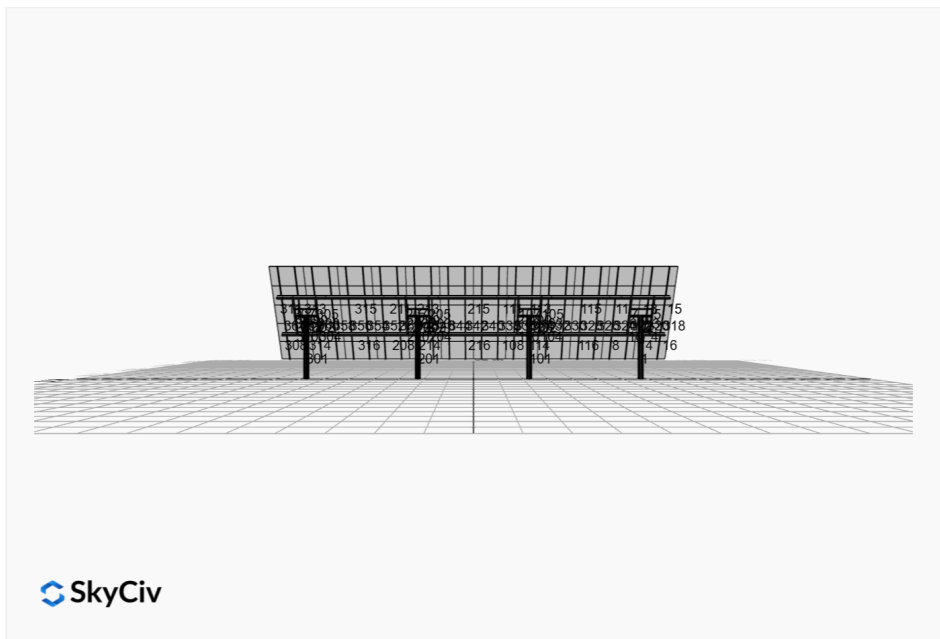
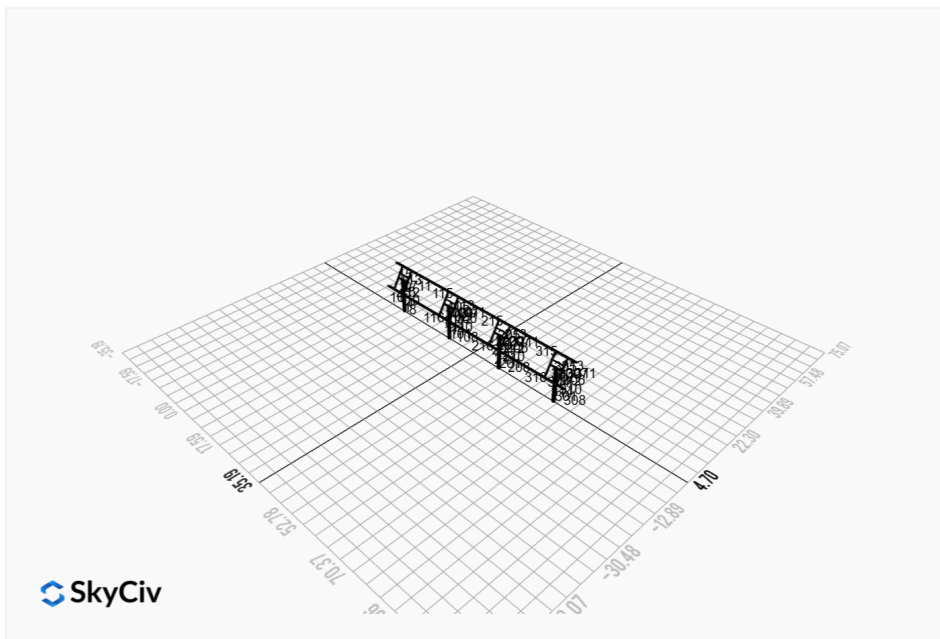
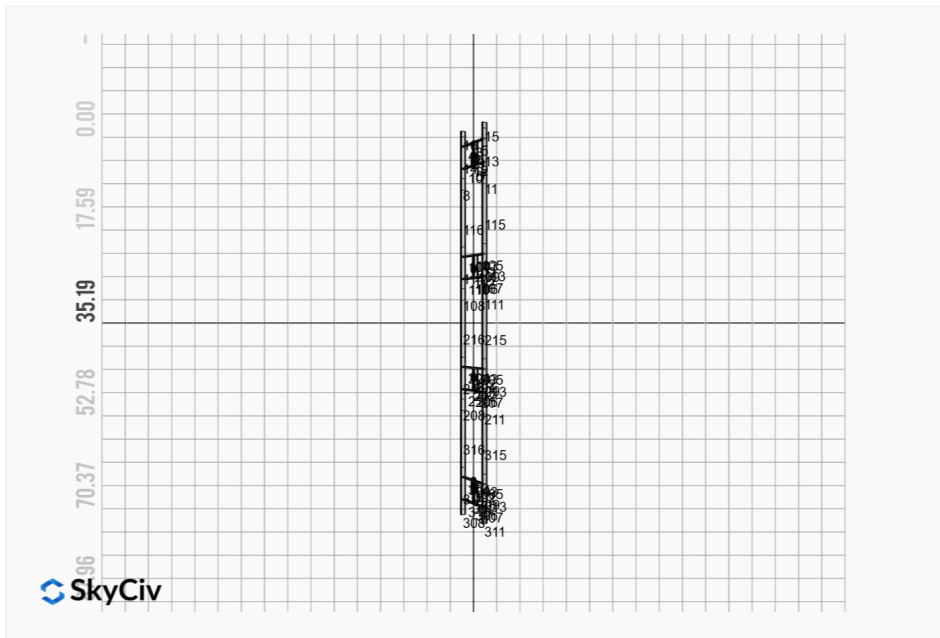
AutoDesigner Input

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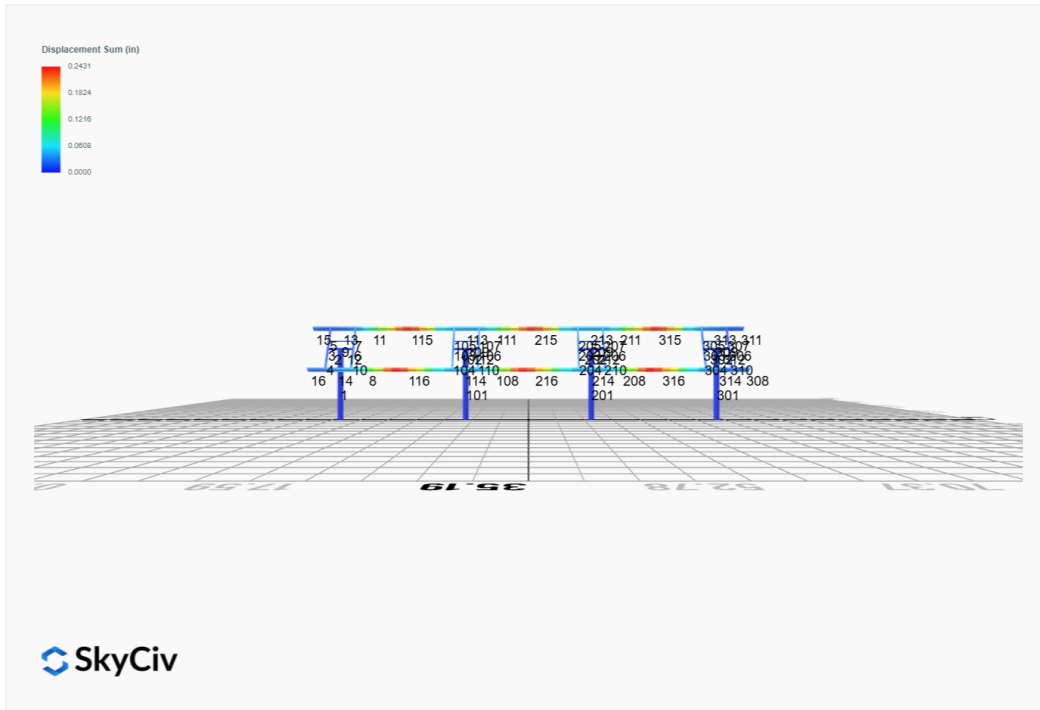
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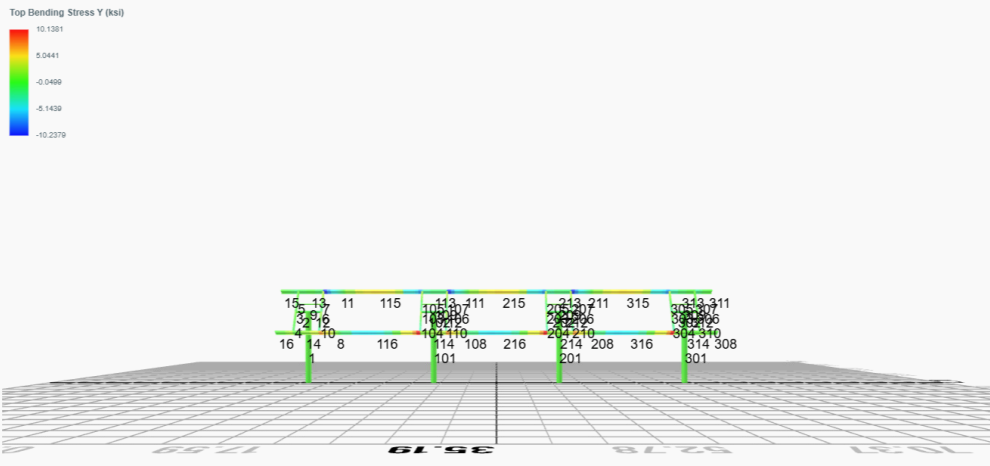
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



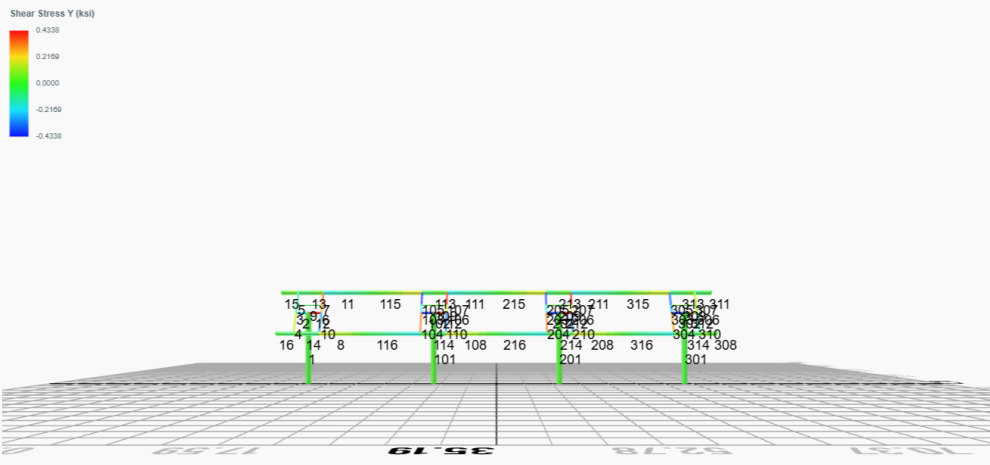


FEM Results (Envelope Worst Case for each member)

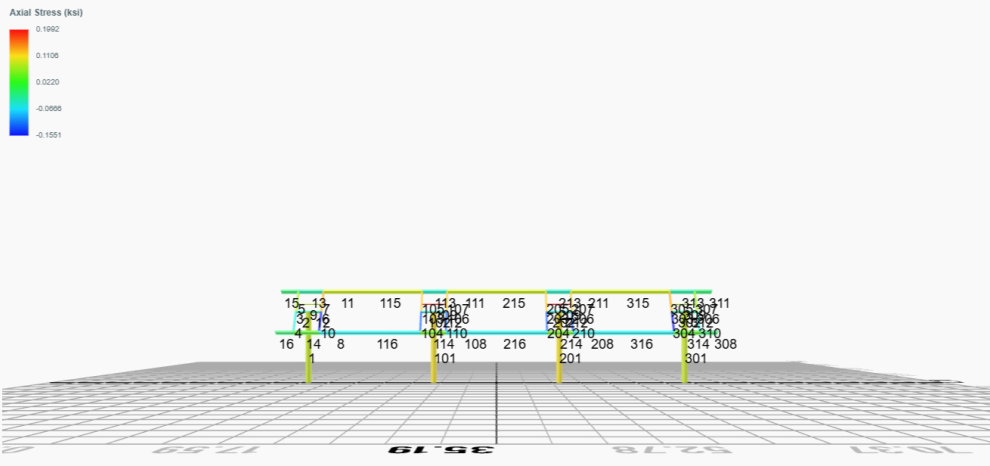




 SkyCiv



 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	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 2. D + L	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 3. D + (S or Lr or R)	0.0746	4.1483	0.1856	0.4727	-0.1589	-0.7918
ULS: 3. D + (S or Lr or R)	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0655	3.7605	0.1628	0.4145	-0.1393	-0.6926
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 5b. D + 0.7E	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0655	3.7605	0.1628	0.4145	-0.1393	-0.6926
ULS: 8. 0.6D + 0.7E	0.0228	1.5584	0.0566	0.1440	-0.0484	-0.2371
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.0987	5.5364	0.3682	0.7773	-3.3017	57.4057
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 5a. D + 0.6W_Wind uplift Case A only	5.1726	-0.3406	-0.1769	-0.2928	3.1166	-57.6894
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.7870	5.9649	0.3681	0.8175	-2.5551	42.6580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0655	3.7605	0.1628	0.4145	-0.1393	-0.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.9165	1.5571	-0.0407	0.0150	2.2586	-43.6634
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0655	3.7605	0.1628	0.4145	-0.1393	-0.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8145	4.8016	0.2997	0.6429	-2.4964	42.9555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.8889	0.3939	-0.1091	-0.1596	2.3173	-43.3659
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0380	2.5973	0.0944	0.2400	-0.0806	-0.3951
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.1139	4.4975	0.3304	0.6813	-3.2694	57.5637
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0228	1.5584	0.0566	0.1440	-0.0484	-0.2371
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	5.1574	-1.3795	-0.2147	-0.3888	3.1488	-57.5314
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0228	1.5584	0.0566	0.1440	-0.0484	-0.2371

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	8.7912
Shear X	-8.6224
Shear Z	0.6167
Moment X	1.3037
Moment Y (Twist)	5.5130
Moment Z	96.5345

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	5.9649
Shear X	-5.1726
Shear Z	0.3682
Moment X	0.8175
Moment Y (Twist)	3.3017
Moment Z	57.6894

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	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 2. D + L	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 3. D + (S or Lr or R)	-0.0746	5.3620	-0.0037	-0.0143	0.0516	0.8355
ULS: 3. D + (S or Lr or R)	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0655	4.8253	-0.0032	-0.0125	0.0453	0.7347

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 5b. D + 0.7E	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0655	4.8253	-0.0032	-0.0125	0.0453	0.7347
ULS: 8. 0.6D + 0.7E	-0.0228	1.9291	-0.0011	-0.0044	0.0157	0.2593
ULS: 5a. D + 0.6W_Wind downforce Case A only	-7.4566	7.5248	0.0111	0.0099	-0.2038	83.3113
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 5a. D + 0.6W_Wind uplift Case A only	7.3826	-1.0957	-0.0138	-0.0226	0.2436	-81.4856
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.6294	8.0575	0.0065	0.0003	-0.1273	62.8940
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0655	4.8253	-0.0032	-0.0125	0.0453	0.7347
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5000	1.5921	-0.0121	-0.0240	0.2083	-60.7037
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0655	4.8253	-0.0032	-0.0125	0.0453	0.7347
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.6019	6.4474	0.0079	0.0056	-0.1463	62.5916
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5275	-0.0180	-0.0108	-0.0187	0.1893	-61.0062
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0380	3.2152	-0.0019	-0.0073	0.0262	0.4322
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-7.4414	6.2387	0.0119	0.0128	-0.2143	83.1384
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0228	1.9291	-0.0011	-0.0044	0.0157	0.2593
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	7.3978	-2.3818	-0.0130	-0.0196	0.2331	-81.6585
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0228	1.9291	-0.0011	-0.0044	0.0157	0.2593

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	12.1139
Shear X	-12.4273
Shear Z	-0.0236
Moment X	-0.0384
Moment Y (Twist)	0.4140
Moment Z	139.5324

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	8.0575
Shear X	-7.4566
Shear Z	-0.0138
Moment X	-0.0240
Moment Y (Twist)	0.2436
Moment Z	83.3113

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	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 2. D + L	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 3. D + (S or Lr or R)	-0.0746	5.3620	0.0037	0.0143	-0.0515	0.8355
ULS: 3. D + (S or Lr or R)	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0655	4.8253	0.0032	0.0125	-0.0452	0.7347
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 5b. D + 0.7E	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0655	4.8253	0.0032	0.0125	-0.0452	0.7347
ULS: 8. 0.6D + 0.7E	-0.0228	1.9291	0.0011	0.0044	-0.0157	0.2593
ULS: 5a. D + 0.6W_Wind downforce Case A only	-7.4566	7.5248	-0.0111	-0.0099	0.2039	83.3113
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 5a. D + 0.6W_Wind uplift Case A only	7.3826	-1.0957	0.0138	0.0226	-0.2436	-81.4856
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322

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	-5.6294	8.0575	-0.0065	-0.0003	0.1274	62.8940
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0655	4.8253	0.0032	0.0125	-0.0452	0.7347
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5000	1.5921	0.0121	0.0240	-0.2082	-60.7037
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0655	4.8253	0.0032	0.0125	-0.0452	0.7347
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.6019	6.4474	-0.0079	-0.0056	0.1464	62.5916
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5275	-0.0180	0.0108	0.0188	-0.1892	-61.0062
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0380	3.2152	0.0019	0.0073	-0.0262	0.4322
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-7.4414	6.2387	-0.0119	-0.0128	0.2144	83.1384
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0228	1.9291	0.0011	0.0044	-0.0157	0.2593
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	7.3978	-2.3818	0.0130	0.0196	-0.2331	-81.6585
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0228	1.9291	0.0011	0.0044	-0.0157	0.2593

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	12.1140
Shear X	-12.4273
Shear Z	0.0236
Moment X	0.0390
Moment Y (Twist)	0.4146
Moment Z	139.5325

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	8.0575
Shear X	-7.4566
Shear Z	0.0138
Moment X	0.0240
Moment Y (Twist)	0.2436
Moment Z	83.3113

Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 2. D + L	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 3. D + (S or Lr or R)	0.0746	4.1483	-0.1856	-0.4730	0.1590	-0.7917
ULS: 3. D + (S or Lr or R)	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0655	3.7605	-0.1628	-0.4148	0.1394	-0.6926
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 5b. D + 0.7E	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0655	3.7605	-0.1628	-0.4148	0.1394	-0.6926
ULS: 8. 0.6D + 0.7E	0.0228	1.5584	-0.0566	-0.1441	0.0484	-0.2371
ULS: 5a. D + 0.6W_Wind downforce Case A only	-5.0987	5.5364	-0.3682	-0.7774	3.3017	57.4057
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 5a. D + 0.6W_Wind uplift Case A only	5.1726	-0.3406	0.1769	0.2927	-3.1165	-57.6894
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.7870	5.9649	-0.3682	-0.8177	2.5552	42.6580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0655	3.7605	-0.1628	-0.4148	0.1394	-0.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.9164	1.5571	0.0407	-0.0152	-2.2585	-43.6633
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0655	3.7605	-0.1628	-0.4148	0.1394	-0.6926
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-3.8145	4.8016	-0.2998	-0.6431	2.4964	42.9555
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	3.8889	0.3939	0.1091	0.1595	-2.3172	-43.3658
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0380	2.5973	-0.0944	-0.2401	0.0806	-0.3951

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-5.1139	4.4975	-0.3304	-0.6813	3.2694	57.5638
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0228	1.5584	-0.0566	-0.1441	0.0484	-0.2371
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	5.1574	-1.3795	0.2147	0.3887	-3.1488	-57.5314
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0228	1.5584	-0.0566	-0.1441	0.0484	-0.2371

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.7912
Shear X	-8.6225
Shear Z	-0.6167
Moment X	-1.3045
Moment Y (Twist)	5.5138
Moment Z	96.5355

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.
 Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	5.9649
Shear X	-5.1726
Shear Z	-0.3682
Moment X	-0.8177
Moment Y (Twist)	3.3017
Moment Z	57.6894

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Sajani Inampudi 60 Modules
 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)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
12	10in Pipe Sch 80	10.75	0.59				

ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

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

212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	97.43	32.46	6.46	56.26	44.91
214	159.30	97.43	32.38	6.46	56.26	44.91
215	159.30	75.13	21.46	6.46	56.26	44.91
216	159.30	75.13	22.19	6.46	56.26	44.91
301	851.50	545.22	229.67	229.67	255.45	255.45
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	137.23	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	137.23	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	37.25	6.46	56.26	44.91
314	159.30	97.43	42.98	6.46	56.26	44.91
315	159.30	75.13	20.71	6.46	56.26	44.91
316	159.30	75.13	20.70	6.46	56.26	44.91

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	0.016	0.420	0.024	0.034	0.002	0.432	#13	0.390	Not Required	Pass
2	0.002	0.127	0.179	0.035	0.043	0.307	#13	0.054	Not Required	Pass
3	0.004	0.393	0.021	0.037	0.002	0.396	#13	0.046	Not Required	Pass
4	0.004	0.393	0.052	0.039	0.012	0.446	#13	0.082	Not Required	Pass
5	0.004	0.245	0.028	0.039	0.008	0.248	#13	0.076	Not Required	Pass
6	0.010	0.667	0.093	0.068	0.027	0.735	#13	0.046	Not Required	Pass
7	0.010	0.413	0.148	0.066	0.036	0.435	#13	0.076	Not Required	Pass
8	0.005	0.141	0.165	0.038	0.018	0.191	#13	0.102	Not Required	Pass
9	0.005	0.060	0.099	0.004	0.005	0.132	#13	0.206	Not Required	Pass
10	0.010	0.637	0.146	0.063	0.033	0.661	#13	0.082	Not Required	Pass
11	0.006	0.134	0.168	0.040	0.018	0.190	#21	0.102	Not Required	Pass
12	0.001	0.332	0.382	0.070	0.074	0.715	#13	0.054	Not Required	Pass
13	0.009	0.075	0.423	0.053	0.023	0.444	#23	0.306	Not Required	Pass
14	0.005	0.067	0.417	0.050	0.023	0.426	#23	0.204	Not Required	Pass
15	0.000	0.004	0.010	0.007	0.003	0.013	#21	Not Required	Not Required	Pass
16	0.000	0.004	0.010	0.007	0.003	0.013	#21	Not Required	Not Required	Pass
101	0.022	0.608	0.001	0.049	0.000	0.619	#13	0.390	Not Required	Pass
102	0.003	0.326	0.410	0.075	0.081	0.738	#13	0.036	Not Required	Pass
103	0.009	0.743	0.057	0.074	0.008	0.783	#13	0.046	Not Required	Pass
104	0.009	0.762	0.152	0.076	0.033	0.832	#13	0.082	Not Required	Pass
105	0.009	0.461	0.157	0.073	0.040	0.487	#13	0.076	Not Required	Pass
106	0.009	0.765	0.057	0.076	0.009	0.801	#13	0.046	Not Required	Pass
107	0.009	0.476	0.153	0.076	0.040	0.501	#13	0.076	Not Required	Pass
108	0.005	0.050	0.156	0.043	0.018	0.183	#21	0.102	Not Required	Pass
109	0.014	0.038	0.070	0.001	0.000	0.112	#13	0.206	Not Required	Pass
110	0.000	0.766	0.146	0.076	0.022	0.828	#13	0.082	Not Required	Pass

110	0.009	0.700	0.140	0.070	0.032	0.020	#13	0.002	Not Required	Pass
111	0.006	0.071	0.162	0.043	0.018	0.179	#21	0.102	Not Required	Pass
112	0.003	0.333	0.423	0.075	0.084	0.757	#13	0.036	Not Required	Pass
113	0.009	0.194	0.427	0.059	0.023	0.533	#21	0.306	Not Required	Pass
114	0.007	0.235	0.422	0.061	0.024	0.546	#21	0.306	Not Required	Pass
115	0.011	0.413	0.228	0.046	0.018	0.555	#13	0.507	Not Required	Pass
116	0.005	0.394	0.226	0.049	0.018	0.535	#13	0.507	Not Required	Pass
201	0.022	0.608	0.001	0.049	0.000	0.619	#13	0.390	Not Required	Pass
202	0.003	0.333	0.423	0.075	0.084	0.757	#13	0.036	Not Required	Pass
203	0.009	0.765	0.057	0.076	0.009	0.801	#13	0.046	Not Required	Pass
204	0.009	0.766	0.146	0.076	0.032	0.828	#13	0.082	Not Required	Pass
205	0.009	0.476	0.153	0.076	0.040	0.501	#13	0.076	Not Required	Pass
206	0.009	0.743	0.057	0.074	0.008	0.783	#13	0.046	Not Required	Pass
207	0.009	0.461	0.157	0.073	0.040	0.487	#13	0.076	Not Required	Pass
208	0.005	0.086	0.168	0.049	0.018	0.181	#21	0.102	Not Required	Pass
209	0.014	0.038	0.070	0.001	0.000	0.112	#13	0.206	Not Required	Pass
210	0.009	0.762	0.152	0.076	0.033	0.832	#13	0.082	Not Required	Pass
211	0.006	0.104	0.172	0.046	0.018	0.176	#21	0.102	Not Required	Pass
212	0.003	0.326	0.410	0.075	0.081	0.737	#13	0.036	Not Required	Pass
213	0.009	0.194	0.427	0.059	0.023	0.532	#21	0.306	Not Required	Pass
214	0.007	0.235	0.422	0.061	0.024	0.545	#21	0.306	Not Required	Pass
215	0.011	0.303	0.229	0.043	0.018	0.446	#13	0.507	Not Required	Pass
216	0.005	0.251	0.226	0.043	0.018	0.392	#13	0.507	Not Required	Pass
301	0.016	0.420	0.024	0.034	0.002	0.432	#13	0.390	Not Required	Pass
302	0.001	0.332	0.382	0.070	0.074	0.715	#13	0.054	Not Required	Pass
303	0.009	0.667	0.093	0.068	0.027	0.735	#13	0.046	Not Required	Pass
304	0.010	0.637	0.146	0.063	0.033	0.661	#13	0.082	Not Required	Pass
305	0.010	0.413	0.148	0.066	0.036	0.435	#13	0.076	Not Required	Pass
306	0.004	0.393	0.021	0.037	0.002	0.396	#13	0.046	Not Required	Pass
307	0.004	0.245	0.028	0.039	0.008	0.248	#13	0.076	Not Required	Pass
308	0.000	0.004	0.010	0.007	0.003	0.013	#21	Not Required	Not Required	Pass
309	0.005	0.060	0.099	0.004	0.005	0.132	#13	0.206	Not Required	Pass
310	0.004	0.393	0.052	0.039	0.012	0.446	#13	0.082	Not Required	Pass
311	0.000	0.004	0.010	0.007	0.003	0.013	#21	Not Required	Not Required	Pass
312	0.002	0.127	0.179	0.035	0.043	0.307	#13	0.054	Not Required	Pass
313	0.009	0.075	0.423	0.053	0.023	0.444	#23	0.204	Not Required	Pass
314	0.005	0.067	0.418	0.050	0.023	0.426	#23	0.306	Not Required	Pass
315	0.011	0.427	0.229	0.040	0.018	0.567	#13	0.507	Not Required	Pass
316	0.005	0.421	0.225	0.038	0.018	0.559	#13	0.507	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis

S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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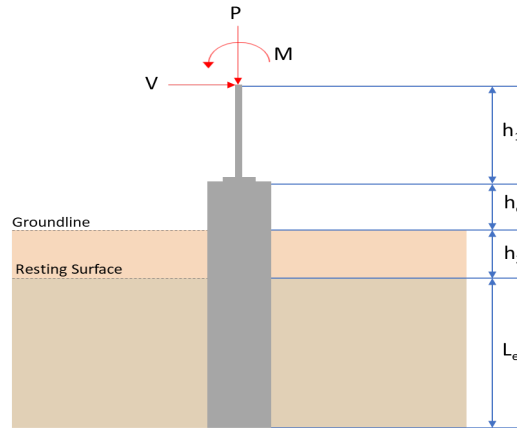
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.965	8.791
V_x (kip)	-5.173	-8.622
V_z (kip)	0.368	0.617
M_x (kipft)	0.817	1.304
M_z (kipft)	57.689	96.534

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,

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

$$H_o = \frac{(-5.173 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.82373 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.689 \text{ kipft}) + ((-5.173 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.1861 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.2282 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.058599 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.817 \text{ kipft}) + ((0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1301 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.7118 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.2282 \text{ ft}), (2.7118 \text{ ft})]$$

$$L_{e,req} = 7.228 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.228 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.9035$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.965 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37281 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18641$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.82373 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.1861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.1861 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (9.1861 \text{ kipft/ft})) + (4 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.549 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.1861 \text{ kipft/ft})) + (3 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (9.1861 \text{ kipft/ft})) + (2 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.23485 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.1861 \text{ kipft/ft})) + ((-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1046 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.549 \text{ ft})}{2}$$

$$p_a = 0.41618 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23485 \text{ kip/ft}^2)}{(0.41618 \text{ kip/ft}^2)}$$

$$Ratio = 0.5643$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1046 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.92051$$

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

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = 0.058599 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.1301 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1301 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.058599 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.1301 \text{ kipft/ft})) + (4 \times (0.058599 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.8041 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.1301 \text{ kipft/ft})) + (3 \times (0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.1301 \text{ kipft/ft})) + (2 \times (0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.032763 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.1301 \text{ kipft/ft})) + ((0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.068342 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8041 \text{ ft})}{2}$$

$$p_a = 0.4353 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.032763 \text{ kip/ft}^2)}{(0.4353 \text{ kip/ft}^2)}$$

$$Ratio = 0.075264$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

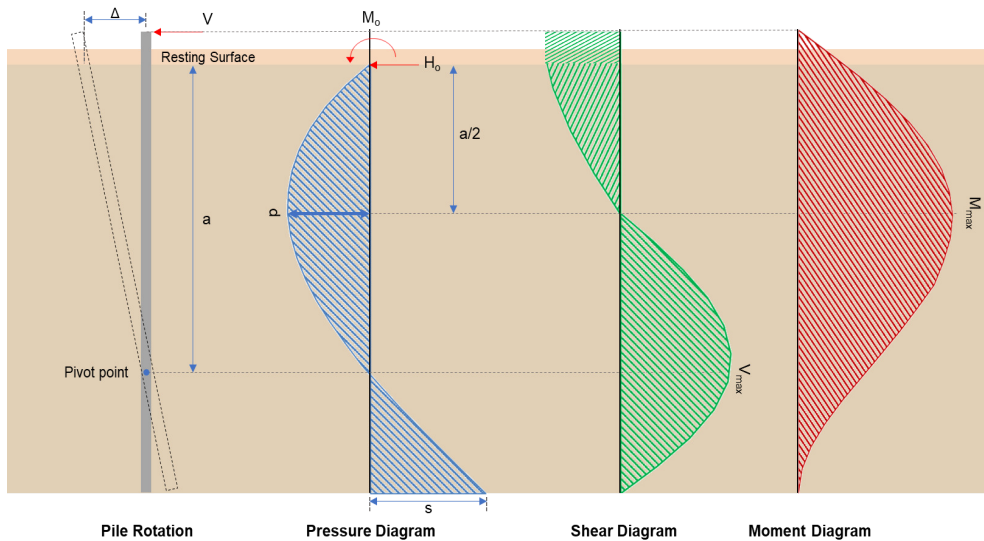
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.068342 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.056952$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-8.622 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3729 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(96.534 \text{ kipft}) + ((-8.622 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.372 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.372 \text{ kipft/ft})}{(-1.3729 \text{ kip/ft})}$$

$$E = 11.196 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.372 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.372 \text{ kipft/ft})) + (4 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.372 \text{ kipft/ft})) + (4 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5484 \text{ ft}$$

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

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

$$V_{max} = ((-1.3729 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5484 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5484 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.221 \text{ kip}$$

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

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

$$M_{max} = ((-1.3729 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(11.196 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 64.907 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.617 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.098248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.304 \text{ kipft}) + ((0.617 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20764 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20764 \text{ kipft/ft})}{(0.098248 \text{ kip/ft})}$$

$$E = 2.1135 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20764 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.098248 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.20764 \text{ kipft/ft})) + (4 \times (0.098248 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.8108 \text{ ft}$$

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

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

$$V_{max} = ((0.098248 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.8108 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.8108 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44812 \text{ kip}$$

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

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

$$M_{max} = ((0.098248 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.1135 \text{ ft})}{(8 \text{ ft})} + \frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5142 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.791 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.304 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.304 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.791 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0032861$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.791 \text{ kip} \rightarrow 8791 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8791 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

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

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

Considering x-direction:

V_{max} = 17.221 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(17.221 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.15534$$

Considering z-direction:

$V_{max} = 0.44812 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.44812 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.0040422$$

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

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:

$M_{max} = 64.907 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(64.907 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.26005$$

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

Considering z-direction:

$M_{max} = 1.5142 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.5142 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0060664$$

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

REFERENCES	CALCULATIONS	RESULTS
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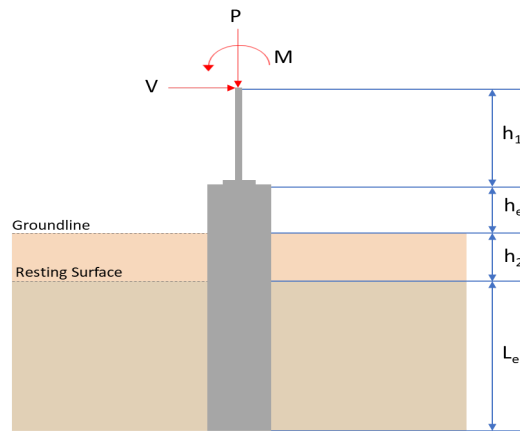
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.965	8.791
V_x (kip)	-5.173	-8.622
V_z (kip)	-0.368	-0.617
M_x (kipft)	-0.818	-1.304
M_z (kipft)	57.689	96.535

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,

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

$$H_o = \frac{(-5.173 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.82373 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(57.689 \text{ kipft}) + ((-5.173 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.1861 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.2282 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.368 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.058599 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.818 \text{ kipft}) + ((-0.368 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13025 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6608 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.2282 \text{ ft}), (1.6608 \text{ ft})]$$

$$L_{e,req} = 7.228 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.228 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.9035$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.965 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37281 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37281 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18641$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.82373 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 9.1861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.1861 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (9.1861 \text{ kipft/ft})) + (4 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.549 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (9.1861 \text{ kipft/ft})) + (3 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (9.1861 \text{ kipft/ft})) + (2 \times (-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.23485 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (9.1861 \text{ kipft/ft})) + ((-0.82373 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1046 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.549 \text{ ft})}{2}$$

$$p_a = 0.41618 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23485 \text{ kip/ft}^2)}{(0.41618 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.5643$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1046 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92051$$

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

Status: **PASS**
Ratio: **0.920**

Considering z-direction:

$H_o = -0.058599 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.13025 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.13025 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.13025 \text{ kipft/ft})) + (4 \times (-0.058599 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.8039 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.13025 \text{ kipft/ft})) + (3 \times (-0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.13025 \text{ kipft/ft})) + (2 \times (-0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.016799 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.13025 \text{ kipft/ft})) + ((-0.058599 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = -0.019526 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8039 \text{ ft})}{2}$$

$$p_a = 0.43529 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.016799 \text{ kip/ft}^2)}{(0.43529 \text{ kip/ft}^2)}$$

$$Ratio = -0.038591$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.019526 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = -0.016272$$

Status: **PASS**
Ratio: **-0.040**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-8.622 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.3729 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(96.535 \text{ kipft}) + ((-8.622 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.372 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(15.372 \text{ kipft/ft})}{(-1.3729 \text{ kip/ft})}$$

$$E = 11.196 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.372 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (15.372 \text{ kipft/ft})) + (4 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (15.372 \text{ kipft/ft})) + (4 \times (-1.3729 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5484 \text{ ft}$$

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

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

$$V_{max} = ((-1.3729 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5484 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5484 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.221 \text{ kip}$$

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

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

$$M_{max} = ((-1.3729 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(11.196 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.196 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5484 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 64.908 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.617 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.098248 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.304 \text{ kipft}) + ((-0.617 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20764 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20764 \text{ kipft/ft})}{(-0.098248 \text{ kip/ft})}$$

$$E = 2.1135 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20764 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.098248 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.20764 \text{ kipft/ft})) + (4 \times (-0.098248 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.8108 \text{ ft}$$

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

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

$$V_{max} = ((-0.098248 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.8108 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.8108 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44812 \text{ kip}$$

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

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

$$M_{max} = ((-0.098248 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(2.1135 \text{ ft})}{(8 \text{ ft})} + \frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1135 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.8108 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5142 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.791 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.304 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.304 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(8.791 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0032861$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.791 \text{ kip} \rightarrow 8791 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(8791 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

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

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

Considering x-direction:

V_{max} = 17.221 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(17.221 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.15535$$

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

Considering z-direction:

$V_{max} = 0.44812 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.44812 \text{ kip})}{(110.86 \text{ kip})}$$

$$Ratio = 0.0040422$$

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:

$M_{max} = 64.908 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(64.908 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.26005$$

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

Considering z-direction:

$M_{max} = 1.5142 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.5142 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0060664$$

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

REFERENCES	CALCULATIONS	RESULTS
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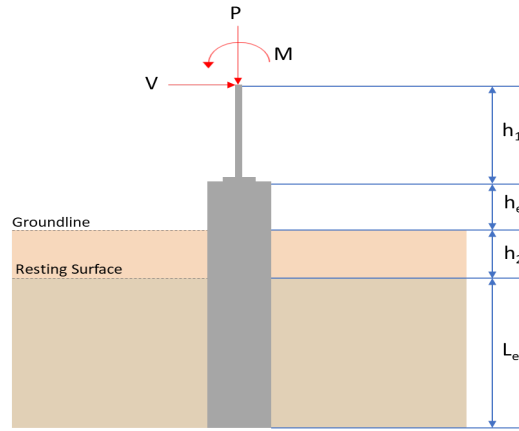
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 8.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.057	12.114
V_x (kip)	-7.457	-12.427
V_z (kip)	-0.014	-0.024
M_x (kipft)	-0.024	-0.038
M_z (kipft)	83.311	139.532

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,

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

$$H_o = \frac{(-7.457 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1874 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(83.311 \text{ kipft}) + ((-7.457 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 13.266 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.9209 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.024 \text{ kipft}) + ((-0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0038217 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.60759 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9209 \text{ ft}), (0.60759 \text{ ft})]$$

$$L_{e,req} = 7.921 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.921 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.90526$$

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

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

$$q = \frac{(8.057 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.50356 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.50356 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25178$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.1874 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 13.266 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.266 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (13.266 \text{ kipft/ft})) + (4 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0835 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (13.266 \text{ kipft/ft})) + (3 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (13.266 \text{ kipft/ft})) + (2 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.24691 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (13.266 \text{ kipft/ft})) + ((-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.265 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0835 \text{ ft})}{2}$$

$$p_a = 0.45626 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24691 \text{ kip/ft}^2)}{(0.45626 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.54117$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.265 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96383$$

Status: **PASS**
Ratio: **0.540**

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = -0.0022293 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0038217 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0038217 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.0038217 \text{ kipft/ft})) + (4 \times (-0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.3969 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0038217 \text{ kipft/ft})) + (3 \times (-0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.0038217 \text{ kipft/ft})) + (2 \times (-0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = -0.00066463 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0038217 \text{ kipft/ft})) + ((-0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = -0.00092968 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.3969 \text{ ft})}{2}$$

$$p_a = 0.47977 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00066463 \text{ kip/ft}^2)}{(0.47977 \text{ kip/ft}^2)}$$

$$Ratio = -0.0013853$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00092968 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = -0.00070832$$

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,

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

$$H_o = \frac{(-12.427 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.9788 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(139.53 \text{ kipft}) + ((-12.427 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 22.218 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(22.218 \text{ kipft/ft})}{(-1.9788 \text{ kip/ft})}$$

$$E = 11.228 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (22.218 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.9788 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times 22.218 \text{ kipft/ft}) + (4 \times (-1.9788 \text{ kip/ft}) \times 8.75 \text{ ft})}$$

$$a = \frac{(-1.9788 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (22.218 \text{ kipft/ft})) + (4 \times (-1.9788 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0826 \text{ ft}$$

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

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

$$V_{max} = ((-1.9788 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.0826 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.0826 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 23.193 \text{ kip}$$

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

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

$$M_{max} = ((-1.9788 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(11.228 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 95.207 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.038 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.006051 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.006051 \text{ kipft/ft})}{(-0.0038217 \text{ kip/ft})}$$

$$E = 1.5833 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.006051 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.006051 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.4068 \text{ ft}$$

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

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

$$V_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.5833 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.4068 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (1.5833 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.4068 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.015232 \text{ kip}$$

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

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

$$M_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(1.5833 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.4068 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (1.5833 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.4068 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.5833 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.4068 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.054842 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.114 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.194 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.194 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$</p> <p>$s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(12.114 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0045283$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.114 \text{ kip} \rightarrow 12114 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12114 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.1 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.1 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.1 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.1 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

V_{max} = 23.193 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.193 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.20867$$

Status: **PASS**
Ratio: **0.210**

Considering z-direction:

$V_{max} = 0.015232 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.015232 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.00013705$$

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:

$M_{max} = 95.207 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(95.207 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.38144$$

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

Considering z-direction:

$M_{max} = 0.054842 \text{ kipft}$ - Maximum moment in the z-direction,

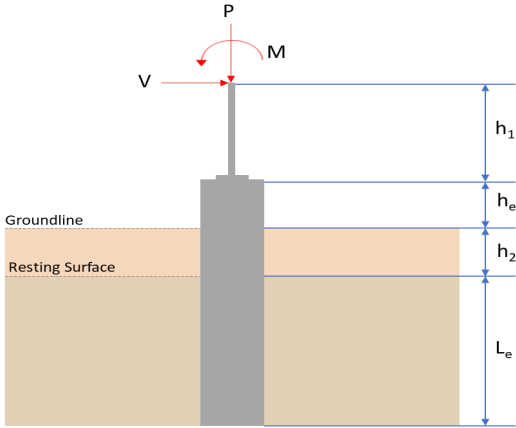
Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.054842 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00021972$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 8.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="368 1088 1225 1189"> <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="652 1290 943 1480"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.057</td> <td>12.114</td> </tr> <tr> <td>V_x (kip)</td> <td>-7.457</td> <td>-12.427</td> </tr> <tr> <td>V_z (kip)</td> <td>0.014</td> <td>0.024</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.024</td> <td>0.039</td> </tr> <tr> <td>M_z (kipft)</td> <td>83.311</td> <td>139.532</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)	8.057	12.114	V_x (kip)	-7.457	-12.427	V_z (kip)	0.014	0.024	M_x (kipft)	0.024	0.039	M_z (kipft)	83.311	139.532	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	83.311	139.532																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-7.457 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -1.1874 \text{ kip/ft}$																											

M_o - Moment per length of pile,

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

$$M_o = \frac{(83.311 \text{ kipft}) + ((-7.457 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 13.266 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.9209 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.014 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0022293 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.024 \text{ kipft}) + ((0.014 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0038217 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.73959 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.9209 \text{ ft}), (0.73959 \text{ ft})]$$

$$L_{e,req} = 7.921 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.921 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.90526$$

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

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

$$q = \frac{(8.057 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.50356 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.50356 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25178$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.1875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.1874 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 13.266 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (13.266 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (13.266 \text{ kipft/ft})) + (4 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0835 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (13.266 \text{ kipft/ft})) + (3 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (13.266 \text{ kipft/ft})) + (2 \times (-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.24691 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (13.266 \text{ kipft/ft})) + ((-1.1874 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.265 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0835 \text{ ft})}{2}$$

$$p_a = 0.45626 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24691 \text{ kip/ft}^2)}{(0.45626 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.54117$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.265 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96383$$

Status: **PASS**
Ratio: **0.540**

Status: **PASS**
Ratio: **0.960**

Considering z-direction:

$H_o = 0.0022293 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0038217 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0038217 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.0038217 \text{ kipft/ft})) + (4 \times (0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.3969 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0038217 \text{ kipft/ft})) + (3 \times (0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.0038217 \text{ kipft/ft})) + (2 \times (0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.0010571 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0038217 \text{ kipft/ft})) + ((0.0022293 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 0.0021276 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.3969 \text{ ft})}{2}$$

$$p_a = 0.47977 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0010571 \text{ kip/ft}^2)}{(0.47977 \text{ kip/ft}^2)}$$

$$Ratio = 0.0022034$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0021276 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0016211$$

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,

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

$$H_o = \frac{(-12.427 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.9788 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(139.53 \text{ kipft}) + ((-12.427 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 22.218 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(22.218 \text{ kipft/ft})}{(-1.9788 \text{ kip/ft})}$$

$$E = 11.228 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (22.218 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-1.9788 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times 22.218 \text{ kipft/ft}) + (4 \times (-1.9788 \text{ kip/ft}) \times 8.75 \text{ ft})}$$

$$a = \frac{(6 \times (22.218 \text{ kipft/ft})) + (4 \times (-1.9788 \text{ kip/ft}) \times (8.75 \text{ ft}))}{(6 \times (22.218 \text{ kipft/ft})) + (4 \times (-1.9788 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0826 \text{ ft}$$

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

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

$$V_{max} = ((-1.9788 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.0826 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.0826 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 23.193 \text{ kip}$$

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

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

$$M_{max} = ((-1.9788 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(11.228 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (11.228 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.0826 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 95.207 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.039 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0062102 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.0062102 \text{ kipft/ft})}{(0.0038217 \text{ kip/ft})}$$

$$E = 1.625 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0062102 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.0062102 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.4036 \text{ ft}$$

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

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

$$V_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.625 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.4036 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.625 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.4036 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.015358 \text{ kip}$$

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

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

$$M_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(1.625 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.4036 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.625 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.4036 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.625 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.4036 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.055389 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.114 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.194 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.194 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(12.114 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0045283$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.114 \text{ kip} \rightarrow 12114 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12114 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.1 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.1 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.1 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((120.1 \text{ kip}) + (50.894 \text{ kip}))$$

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

Considering x-direction:

V_{max} = 23.193 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.193 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.20867$$

Status: **PASS**
Ratio: **0.210**

Considering z-direction:

$V_{max} = 0.015358 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.015358 \text{ kip})}{(111.15 \text{ kip})}$$

$$Ratio = 0.00013818$$

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:

$M_{max} = 95.207 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(95.207 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.38144$$

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

Considering z-direction:

$M_{max} = 0.055389 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.055389 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00022191$$

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