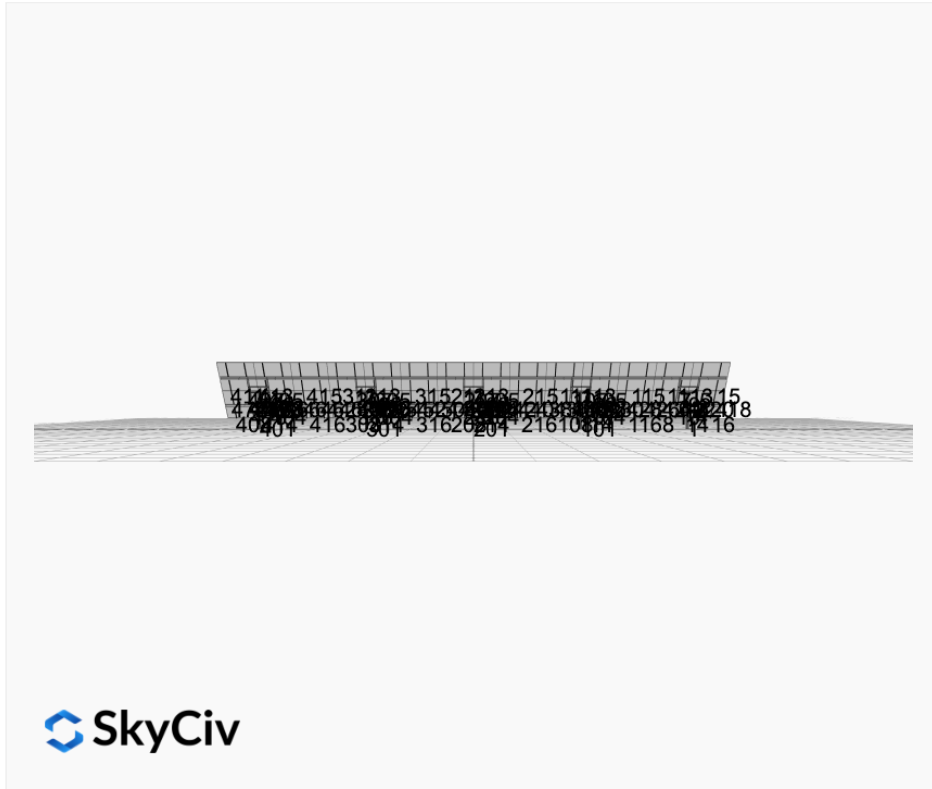


Project Name: MTSOLAR_36ID5CHAEIHF 4x14 - V1Jb
Location: 6848 Lake Elora Dr, Canyon, MN 55717, USA
Unique ID: 5P-22.5-6TOP-SD-45-L-4Hx14W-A4AG
Dealer: _____

Date: Tue Jul 22 2025
Number of Modules: 56
Number of Poles: 5
Date Sold: _____



Array Dimensions N/S	15.05 ft
Array Dimensions E/W	105.84 ft
Winter Tilt Angle	50
Front Edge Clearance	2 ft

MT Solar Bill of Materials (5P-22.5-6TOP-SD-45-L-4Hx14W-A4AG)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	5
MTS-HF-SD	H-Frame Assembly-SD	5
MTS-SD-Wing-45	45IN SD Wing	4
MTS-SD-Splice-90	90IN SD Splice	16
MTS-CLAMP-HOOK-4PK	Hook Clamp	14

Rail Bill of Materials

Part	Qty
Rails (181in)	28
Rail Attachment	56
Module Mid Clamp	84
Module End Clamp	56
Ground Lug	14

Site Details:



Site Address: 6848 Lake Elora Dr, Canyon, MN 55717, USA

Array Specification

Duty Classification:	SD
Module Width:	44.65 in
Module Length:	89.72in
Number of Rows:	4
Number of Columns:	14
Total Number of Modules:	56
Winter Tilt Angle:	50
Front Edge Clearance:	2
Total Array Height at Tilt:	13.53 ft
Total Frame Length:	105.00 ft
Module Info/Notes:	Heliene
Array Dimensions N/S:	15.05 ft
Array Dimensions E/W:	105.84 ft
Rail Length:	180.60 in
Rail Spacing:	3.78 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	7.76 ft
Number of Poles:	5
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.75 ft Pile 3: 5.75 ft Pile 4: 5.75 ft Pile 5: 5.25 ft
Foundation Volume:	16.444 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	6848 Lake Elora Dr, Canyon, MN 55717, USA
Wind Speed:	100 mph

Snow Load:

60 psf

Design Disclaimer

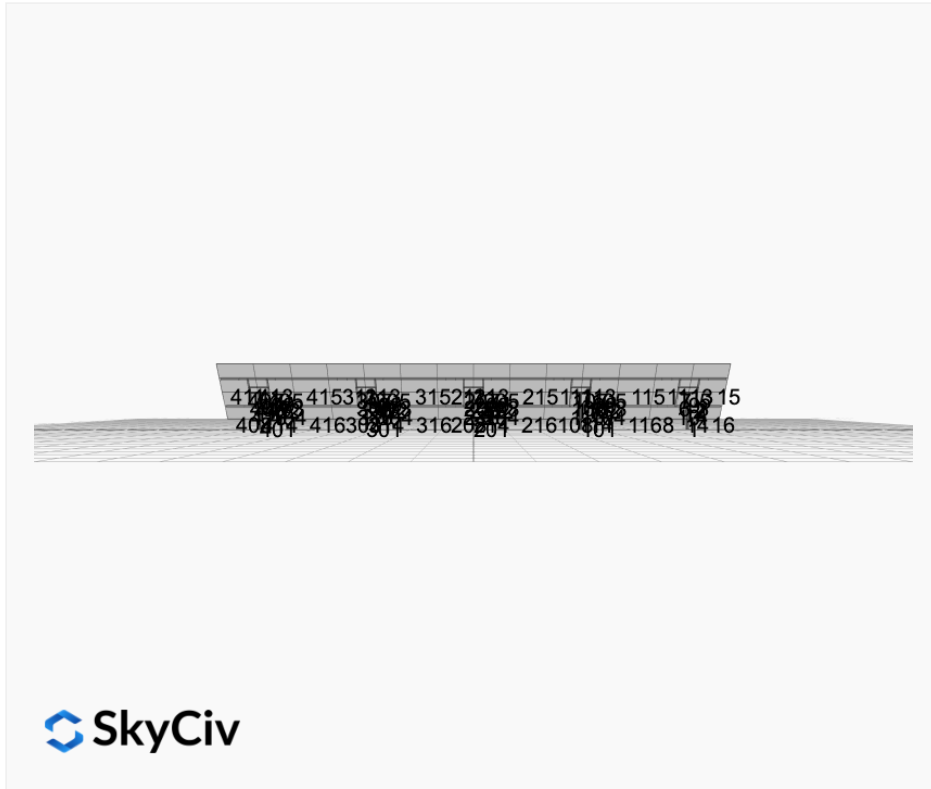
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

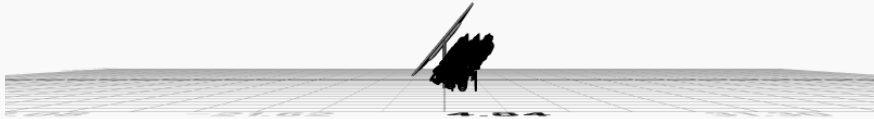
AutoDesigner Input

```
{ "wind_speed_override": null, "snow_load_override": null, "direct_snow_load": false, "add_angle_brace": false, "product_type": "Beam", "designer_name": "Michael Serre", "designer_email": "meserre@aol.com", "designer_phone": "2184815400", "project_id": "MTSOLAR_36ID5CHAEIHF 4x14 - V1Jb", "site_address": "6848 Lake Elora Dr, Canyon, MN 55717, USA", "module_info": "Heliene", "module_width": 44.65, "module_length": 89.72, "number_rows": 4, "number_columns": 14, "pole_mount_section": "4_40", "core_pipe_width": 65, "core_pipe_section": "2_40", "adjuster_section": "2_40", "core_beam_height": 65, "core_beam_section": "HSS3x2x1/8", "main_pipe_section": "2_126A", "pole_spacing": 15, "tilt_angle": 50, "ground_clearance": 2, "risk_category": "I", "exposure_category": "C", "frame_duty_override": "auto", "pole_override": "auto", "soil_type": "sand", "customer_foundation_override": "48_Square", "foundation_type": "Square", "foundation_size": 48, "check_rails": false }
```

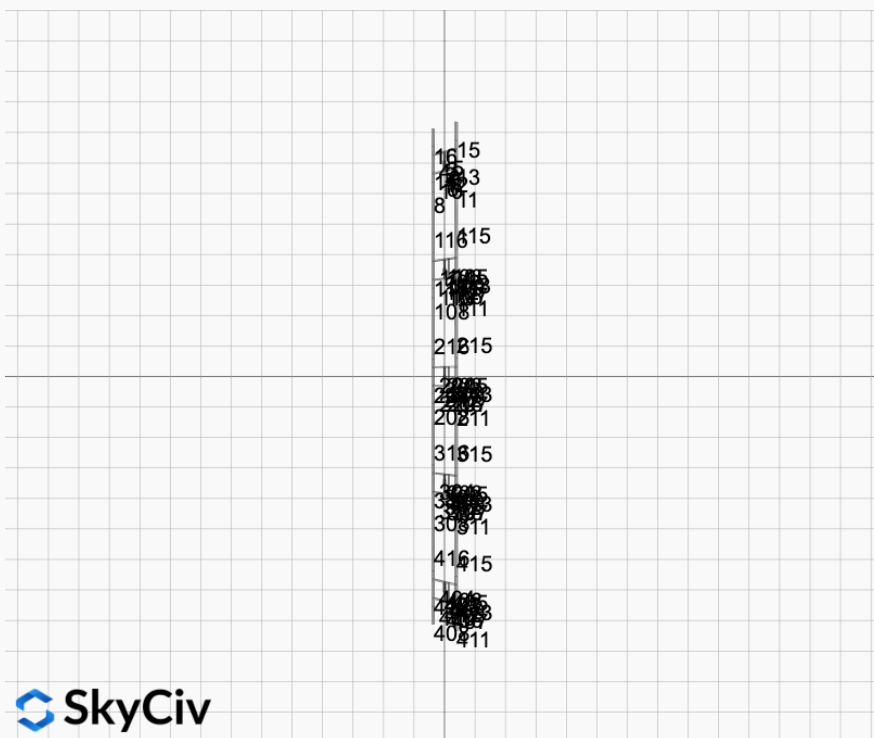
Design Notes:

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

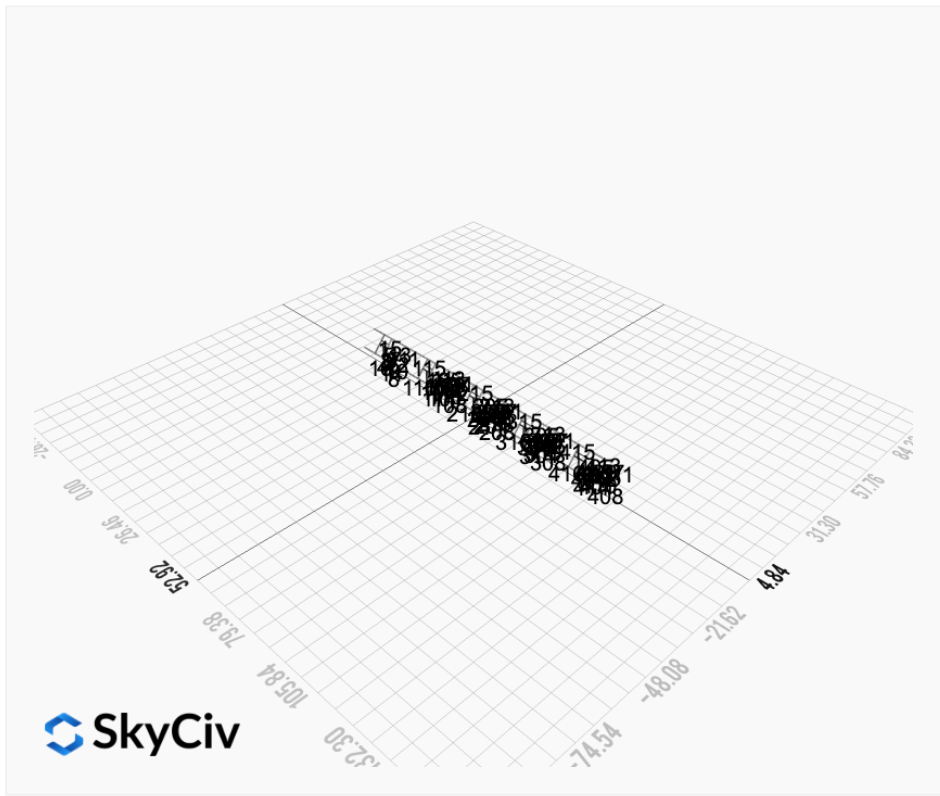




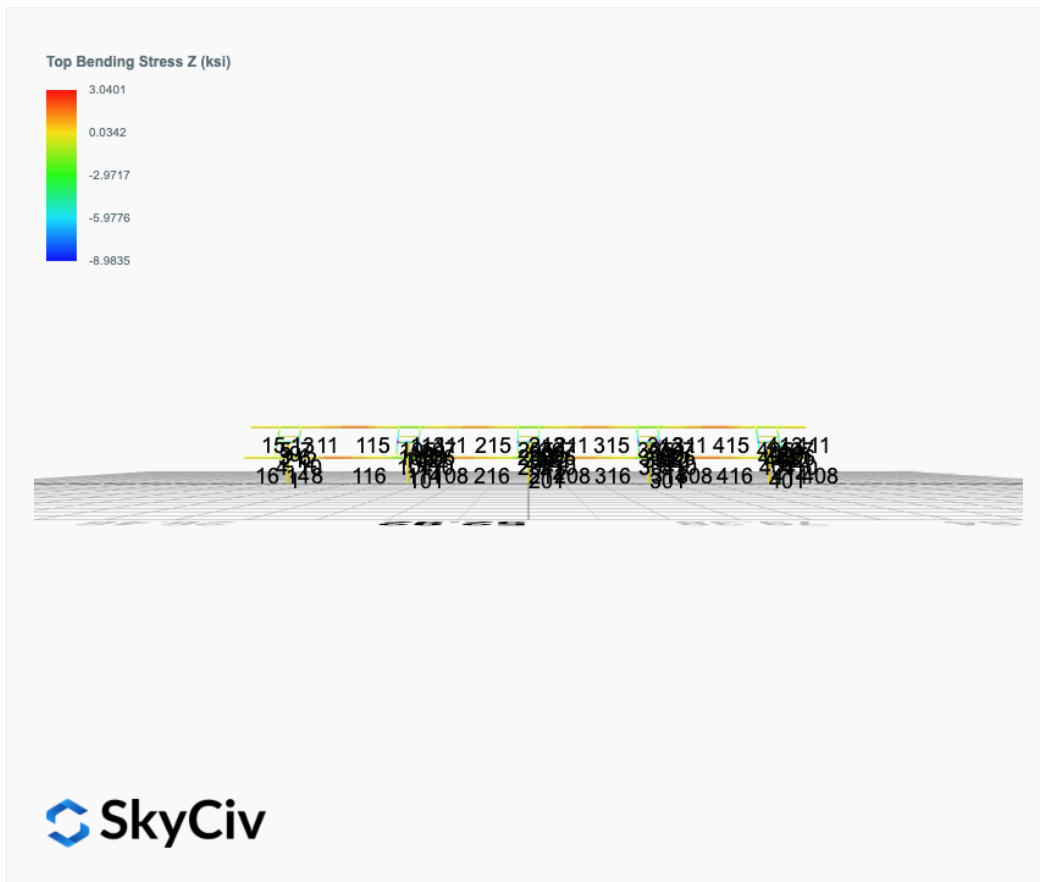
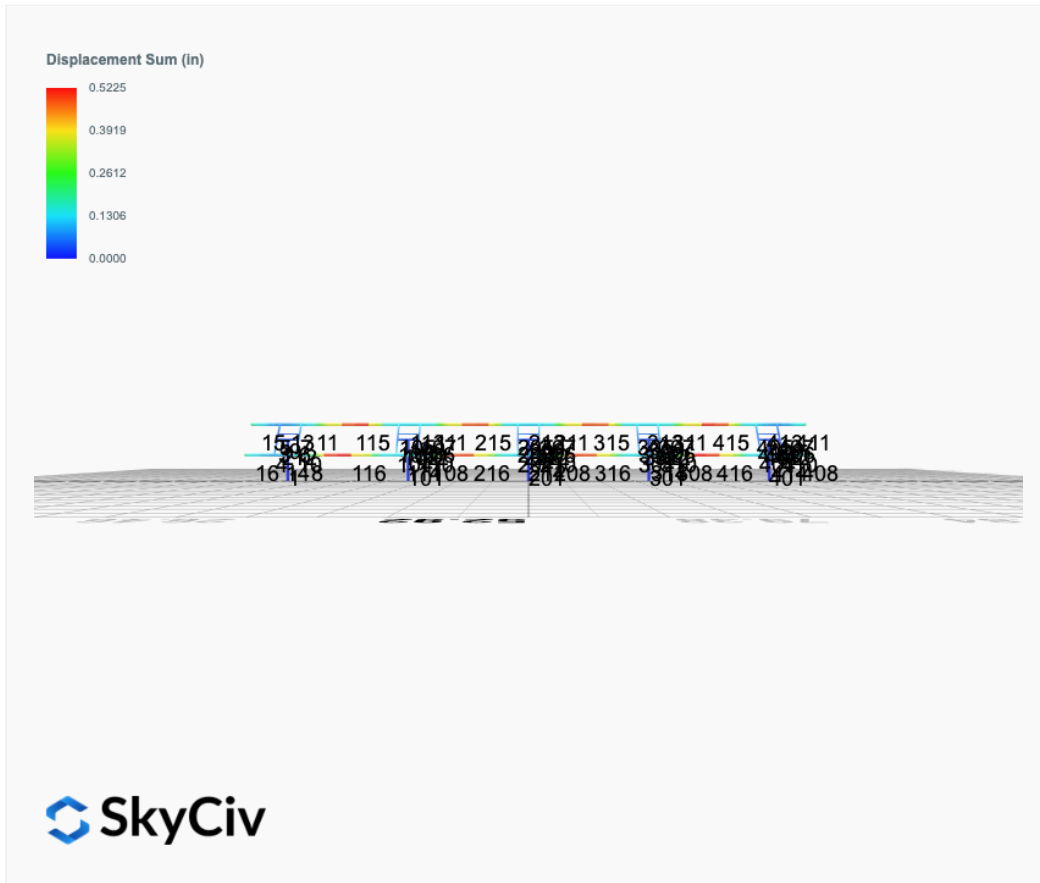
 SkyCiv



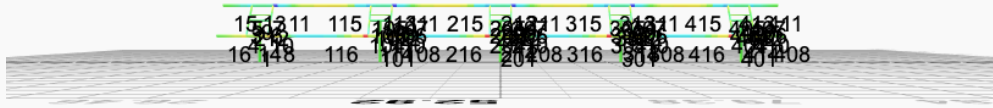
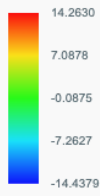
 SkyCiv



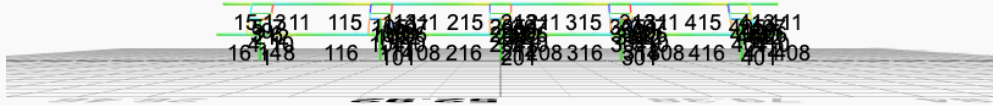
FEM Results (Envelope Worst Case for each member)



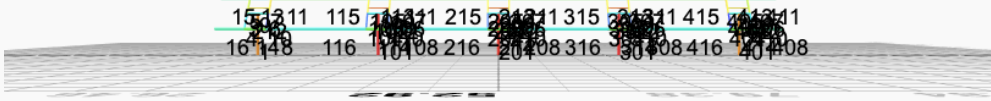
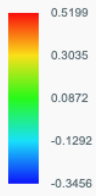
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 2. D + L	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 3. D + (S or Lr or R)	0.0514	4.2754	0.2065	0.4453	-0.2446	-0.3515
ULS: 3. D + (S or Lr or R)	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0437	3.6835	0.1757	0.3789	-0.2082	-0.2977
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 5b. D + 0.7E	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0437	3.6835	0.1757	0.3789	-0.2082	-0.2977
ULS: 8. 0.6D + 0.7E	0.0125	1.1447	0.0501	0.1078	-0.0593	-0.0818
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0987	3.6722	0.2560	0.5166	-0.6673	16.5917
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1395	0.1437	-0.0866	-0.1520	0.4621	-16.5059
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5459	5.0068	0.3051	0.6315	-0.6345	12.2484
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0437	3.6835	0.1757	0.3789	-0.2082	-0.2977
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6328	2.3605	0.0482	0.1301	0.2126	-12.5749
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0437	3.6835	0.1757	0.3789	-0.2082	-0.2977
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5688	3.2311	0.2128	0.4323	-0.5252	12.4097
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6098	0.5847	-0.0441	-0.0691	0.3219	-12.4135
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0208	1.9078	0.0834	0.1797	-0.0988	-0.1363
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1070	2.9091	0.2226	0.4447	-0.6278	16.6463
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0125	1.1447	0.0501	0.1078	-0.0593	-0.0818
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1312	-0.6194	-0.1200	-0.2239	0.5017	-16.4513
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0125	1.1447	0.0501	0.1078	-0.0593	-0.0818

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	7.5478
Shear X	-3.5720
Shear Z	0.4517
Moment X	0.9328
Moment Y (Twist)	1.1450
Moment Z	28.0247

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	5.0068
Shear X	-2.1395
Shear Z	0.3051
Moment X	0.6315
Moment Y (Twist)	0.6673
Moment Z	16.6463

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	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 2. D + L	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 3. D + (S or Lr or R)	-0.0559	5.1732	-0.0252	-0.0555	0.0461	0.4271
ULS: 3. D + (S or Lr or R)	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0476	4.4477	-0.0215	-0.0472	0.0392	0.3647

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 5b. D + 0.7E	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0476	4.4477	-0.0215	-0.0472	0.0392	0.3647
ULS: 8. 0.6D + 0.7E	-0.0136	1.3629	-0.0061	-0.0135	0.0112	0.1063
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6566	4.4960	-0.0204	-0.0489	0.0270	20.7699
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6125	0.0463	0.0008	0.0052	0.0070	-19.8985
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0231	6.1162	-0.0291	-0.0670	0.0455	15.8092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0476	4.4477	-0.0215	-0.0472	0.0392	0.3647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9288	2.7789	-0.0132	-0.0265	0.0305	-14.6921
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0476	4.4477	-0.0215	-0.0472	0.0392	0.3647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9981	3.9399	-0.0179	-0.0423	0.0249	15.6217
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9537	0.6026	-0.0020	-0.0017	0.0099	-14.8796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0226	2.2714	-0.0102	-0.0225	0.0187	0.1772
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6476	3.5874	-0.0164	-0.0399	0.0195	20.6990
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0136	1.3629	-0.0061	-0.0135	0.0112	0.1063
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6215	-0.8623	0.0049	0.0142	-0.0005	-19.9694
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0136	1.3629	-0.0061	-0.0135	0.0112	0.1063

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.2220
Shear X	-4.4326
Shear Z	-0.0442
Moment X	-0.1006
Moment Y (Twist)	0.0702
Moment Z	35.2549

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.1162
Shear X	-2.6566
Shear Z	-0.0291
Moment X	-0.0670
Moment Y (Twist)	0.0461
Moment Z	20.7699

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	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 2. D + L	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 3. D + (S or Lr or R)	0.0090	5.1115	-0.0000	-0.0014	-0.0002	0.0044
ULS: 3. D + (S or Lr or R)	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0076	4.3953	-0.0000	-0.0012	-0.0002	0.0049
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 5b. D + 0.7E	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0076	4.3953	-0.0000	-0.0012	-0.0002	0.0049
ULS: 8. 0.6D + 0.7E	0.0022	1.3479	-0.0000	-0.0003	-0.0001	0.0038
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5845	4.4176	-0.0001	-0.0014	-0.0010	20.4329
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5912	0.0760	0.0000	0.0003	0.0008	-19.8945
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9335	6.0236	-0.0000	-0.0018	-0.0009	15.3248
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0076	4.3953	-0.0000	-0.0012	-0.0002	0.0049
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9483	2.7674	0.0000	-0.0005	0.0005	-14.9208
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0076	4.3953	-0.0000	-0.0012	-0.0002	0.0049
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9375	3.8748	-0.0000	-0.0012	-0.0008	15.3263
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9443	0.6186	0.0000	0.0001	0.0006	-14.9193
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0036	2.2465	-0.0000	-0.0005	-0.0001	0.0063
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5860	3.5190	-0.0001	-0.0012	-0.0010	20.4304
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0022	1.3479	-0.0000	-0.0003	-0.0001	0.0038
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5898	-0.8226	0.0001	0.0005	0.0008	-19.8970
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0022	1.3479	-0.0000	-0.0003	-0.0001	0.0038

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.0899
Shear X	-4.3203
Shear Z	-0.0001
Moment X	-0.0027
Moment Y (Twist)	0.0017
Moment Z	34.6579

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.0236
Shear X	-2.5912
Shear Z	-0.0001
Moment X	-0.0018
Moment Y (Twist)	0.0010
Moment Z	20.4329

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	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 2. D + L	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 3. D + (S or Lr or R)	-0.0559	5.1732	0.0251	0.0526	-0.0465	0.4271
ULS: 3. D + (S or Lr or R)	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0476	4.4478	0.0214	0.0448	-0.0396	0.3647
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 5b. D + 0.7E	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0476	4.4478	0.0214	0.0448	-0.0396	0.3647
ULS: 8. 0.6D + 0.7E	-0.0136	1.3629	0.0061	0.0128	-0.0113	0.1063
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.6566	4.4960	0.0203	0.0459	-0.0288	20.7699
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6125	0.0463	-0.0007	-0.0046	-0.0054	-19.8986
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0231	6.1162	0.0290	0.0632	-0.0471	15.8092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0476	4.4478	0.0214	0.0448	-0.0396	0.3647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9288	2.7789	0.0132	0.0253	-0.0295	-14.6922
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0476	4.4478	0.0214	0.0448	-0.0396	0.3647
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9981	3.9399	0.0177	0.0398	-0.0263	15.6217
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9537	0.6026	0.0020	0.0019	-0.0088	-14.8796
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0226	2.2714	0.0102	0.0214	-0.0188	0.1772

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.6476	3.5875	0.0162	0.0374	-0.0213	20.6990
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0136	1.3629	0.0061	0.0128	-0.0113	0.1063
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6215	-0.8623	-0.0048	-0.0132	0.0021	-19.9694
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0136	1.3629	0.0061	0.0128	-0.0113	0.1063

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	9.2220
Shear X	-4.4326
Shear Z	0.0440
Moment X	0.0950
Moment Y (Twist)	0.0721
Moment Z	35.2550

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.1162
Shear X	-2.6566
Shear Z	0.0290
Moment X	0.0632
Moment Y (Twist)	0.0471
Moment Z	20.7699

Reaction Forces for Foundation 5 (Node ID#401), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 2. D + L	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 3. D + (S or Lr or R)	0.0514	4.2753	-0.2064	-0.4478	0.2431	-0.3519
ULS: 3. D + (S or Lr or R)	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0438	3.6834	-0.1756	-0.3810	0.2069	-0.2980
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 5b. D + 0.7E	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0438	3.6834	-0.1756	-0.3810	0.2069	-0.2980
ULS: 8. 0.6D + 0.7E	0.0125	1.1446	-0.0500	-0.1084	0.0589	-0.0819
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.0986	3.6720	-0.2558	-0.5187	0.6643	16.5907
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.1394	0.1437	0.0864	0.1521	-0.4603	-16.5051
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5458	5.0066	-0.3049	-0.6345	0.6315	12.2474
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0438	3.6834	-0.1756	-0.3810	0.2069	-0.2980
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6327	2.3604	-0.0482	-0.1315	-0.2120	-12.5745
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0438	3.6834	-0.1756	-0.3810	0.2069	-0.2980
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.5687	3.2310	-0.2127	-0.4342	0.5228	12.4089
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6098	0.5847	0.0440	0.0689	-0.3207	-12.4130
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0208	1.9077	-0.0834	-0.1807	0.0982	-0.1365
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.1069	2.9089	-0.2224	-0.4464	0.6250	16.6453
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0125	1.1446	-0.0500	-0.1084	0.0589	-0.0819
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.1311	-0.6194	0.1198	0.2243	-0.4996	-16.4505
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0125	1.1446	-0.0500	-0.1084	0.0589	-0.0819

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.

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.5475
Shear X	-3.5718
Shear Z	-0.4513
Moment X	-0.9374
Moment Y (Twist)	1.1398
Moment Z	28.0233

Result	Value (kip, kip-ft)
Axial	5.0066
Shear X	-2.1394
Shear Z	-0.3049
Moment X	-0.6345
Moment Y (Twist)	0.6643
Moment Z	16.6453

Project Details

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

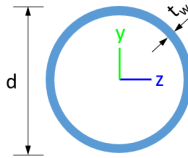


Design Input Information

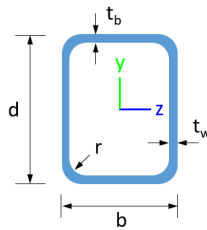
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F _y (ksi)	F _u (ksi)
1	29000	50	65

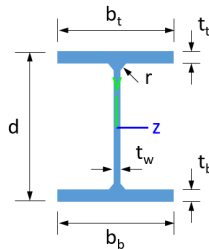
Section Dimensions



ID	Name	d (in)	t _w (in)				
1	2in Pipe Sch 40	2.38	0.15				
4	4in Pipe Sch 40	4.50	0.24				
7	6in Pipe Sch 40	6.63	0.28				



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



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

Section Properties

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

104	79.65	72.84	10.99	6.26	29.14	16.61
105	79.65	74.30	10.99	6.26	29.14	16.61
106	79.65	74.89	10.99	6.26	29.14	16.61
107	79.65	74.30	10.99	6.26	29.14	16.61
108	120.60	115.40	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.84	10.99	6.26	29.14	16.61
111	120.60	115.40	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	84.02	18.19	6.45	30.09	45.74
114	120.60	84.02	18.14	6.45	30.09	45.74
115	120.60	20.58	11.15	6.45	30.09	45.74
116	120.60	20.58	11.50	6.45	30.09	45.74
201	251.16	144.16	42.30	42.30	75.35	75.35
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.89	10.99	6.26	29.14	16.61
204	79.65	72.84	10.99	6.26	29.14	16.61
205	79.65	74.30	10.99	6.26	29.14	16.61
206	79.65	74.89	10.99	6.26	29.14	16.61
207	79.65	74.30	10.99	6.26	29.14	16.61
208	120.60	115.40	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.84	10.99	6.26	29.14	16.61
211	120.60	115.40	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	84.02	18.14	6.45	30.09	45.74
214	120.60	84.02	18.15	6.45	30.09	45.74
215	120.60	20.58	11.42	6.45	30.09	45.74
216	120.60	20.58	11.52	6.45	30.09	45.74
301	251.16	144.16	42.30	42.30	75.35	75.35
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.89	10.99	6.26	29.14	16.61
304	79.65	72.84	10.99	6.26	29.14	16.61
305	79.65	74.30	10.99	6.26	29.14	16.61
306	79.65	74.89	10.99	6.26	29.14	16.61
307	79.65	74.30	10.99	6.26	29.14	16.61
308	120.60	115.40	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.84	10.99	6.26	29.14	16.61
311	120.60	115.40	23.36	6.45	30.09	45.74
312	142.83	140.22	16.17	16.17	42.85	42.85
313	120.60	84.02	18.18	6.45	30.09	45.74
314	120.60	84.02	18.14	6.45	30.09	45.74
315	120.60	20.58	11.35	6.45	30.09	45.74
316	120.60	20.58	11.65	6.45	30.09	45.74
401	251.16	144.16	42.30	42.30	75.35	75.35
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.89	10.99	6.26	29.14	16.61
404	79.65	72.84	10.99	6.26	29.14	16.61
405	79.65	74.30	10.99	6.26	29.14	16.61
406	79.65	74.89	10.99	6.26	29.14	16.61
407	79.65	74.30	10.99	6.26	29.14	16.61

407	79.03	74.30	10.99	0.20	29.14	10.01
408	120.60	54.44	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.84	10.99	6.26	29.14	16.61
411	120.60	54.44	23.36	6.45	30.09	45.74
412	142.83	140.22	16.17	16.17	42.85	42.85
413	120.60	84.02	18.78	6.45	30.09	45.74
414	120.60	84.02	18.90	6.45	30.09	45.74
415	120.60	20.58	11.06	6.45	30.09	45.74
416	120.60	20.58	11.09	6.45	30.09	45.74

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.052	0.663	0.061	0.047	0.006	0.706	#13	0.436	Not Required	Pass
2	0.006	0.270	0.190	0.068	0.038	0.425	#13	0.053	Not Required	Pass
3	0.012	0.488	0.069	0.048	0.009	0.501	#13	0.044	Not Required	Pass
4	0.010	0.489	0.230	0.049	0.040	0.616	#21	0.078	Not Required	Pass
5	0.011	0.303	0.203	0.049	0.041	0.325	#21	0.073	Not Required	Pass
6	0.018	0.661	0.176	0.068	0.029	0.778	#21	0.044	Not Required	Pass
7	0.019	0.411	0.392	0.066	0.078	0.473	#21	0.073	Not Required	Pass
8	0.004	0.088	0.231	0.041	0.019	0.237	#23	0.088	Not Required	Pass
9	0.019	0.052	0.105	0.004	0.005	0.132	#13	0.198	Not Required	Pass
10	0.019	0.637	0.363	0.064	0.061	0.782	#21	0.078	Not Required	Pass
11	0.006	0.078	0.240	0.043	0.019	0.250	#21	0.088	Not Required	Pass
12	0.004	0.425	0.261	0.104	0.049	0.654	#13	0.052	Not Required	Pass
13	0.008	0.167	0.511	0.053	0.024	0.643	#21	0.265	Not Required	Pass
14	0.008	0.160	0.500	0.052	0.024	0.612	#21	0.177	Not Required	Pass
15	0.000	0.054	0.129	0.022	0.010	0.178	#21	Not Required	Not Required	Pass
16	0.000	0.054	0.129	0.022	0.010	0.178	#21	Not Required	Not Required	Pass
101	0.064	0.833	0.006	0.059	0.001	0.863	#13	0.436	Not Required	Pass
102	0.006	0.438	0.291	0.107	0.052	0.689	#13	0.053	Not Required	Pass
103	0.018	0.712	0.113	0.071	0.006	0.782	#13	0.044	Not Required	Pass
104	0.017	0.726	0.363	0.073	0.060	0.894	#21	0.078	Not Required	Pass
105	0.018	0.442	0.393	0.071	0.079	0.511	#21	0.073	Not Required	Pass
106	0.018	0.708	0.112	0.071	0.007	0.771	#13	0.044	Not Required	Pass
107	0.018	0.441	0.389	0.070	0.078	0.503	#21	0.073	Not Required	Pass
108	0.004	0.075	0.229	0.045	0.019	0.298	#21	0.088	Not Required	Pass
109	0.028	0.047	0.071	0.001	0.000	0.125	#13	0.198	Not Required	Pass
110	0.017	0.707	0.369	0.071	0.061	0.886	#21	0.078	Not Required	Pass
111	0.005	0.066	0.236	0.045	0.019	0.301	#21	0.088	Not Required	Pass
112	0.007	0.423	0.288	0.104	0.052	0.675	#13	0.034	Not Required	Pass
113	0.008	0.235	0.512	0.057	0.024	0.719	#21	0.265	Not Required	Pass
114	0.009	0.260	0.504	0.059	0.024	0.726	#21	0.265	Not Required	Pass
115	0.031	0.362	0.279	0.046	0.019	0.613	#21	0.858	Not Required	Pass
116	0.009	0.355	0.276	0.048	0.019	0.585	#21	0.858	Not Required	Pass
201	0.063	0.819	0.000	0.057	0.000	0.846	#13	0.436	Not Required	Pass
202	0.006	0.423	0.282	0.104	0.050	0.667	#13	0.034	Not Required	Pass
203	0.018	0.703	0.112	0.070	0.007	0.771	#13	0.044	Not Required	Pass
204	0.017	0.692	0.361	0.069	0.060	0.865	#21	0.078	Not Required	Pass
205	0.018	0.437	0.386	0.070	0.078	0.500	#21	0.073	Not Required	Pass

206	0.018	0.703	0.112	0.070	0.007	0.771	#13	0.044	Not Required	Pass
207	0.018	0.437	0.386	0.070	0.078	0.500	#21	0.073	Not Required	Pass
208	0.004	0.066	0.225	0.044	0.019	0.285	#21	0.088	Not Required	Pass
209	0.027	0.046	0.069	0.001	0.000	0.122	#13	0.198	Not Required	Pass
210	0.017	0.692	0.361	0.069	0.060	0.865	#21	0.078	Not Required	Pass
211	0.005	0.069	0.234	0.045	0.019	0.299	#21	0.088	Not Required	Pass
212	0.006	0.423	0.282	0.104	0.050	0.667	#13	0.034	Not Required	Pass
213	0.007	0.244	0.503	0.055	0.024	0.719	#21	0.265	Not Required	Pass
214	0.009	0.243	0.493	0.055	0.024	0.699	#21	0.265	Not Required	Pass
215	0.031	0.292	0.275	0.045	0.019	0.551	#21	0.858	Not Required	Pass
216	0.009	0.286	0.273	0.044	0.019	0.531	#21	0.858	Not Required	Pass
301	0.064	0.833	0.006	0.059	0.001	0.863	#13	0.436	Not Required	Pass
302	0.007	0.423	0.288	0.104	0.052	0.674	#13	0.034	Not Required	Pass
303	0.018	0.708	0.112	0.071	0.007	0.771	#13	0.044	Not Required	Pass
304	0.017	0.708	0.369	0.071	0.061	0.886	#21	0.078	Not Required	Pass
305	0.018	0.441	0.389	0.070	0.078	0.504	#21	0.073	Not Required	Pass
306	0.018	0.711	0.113	0.071	0.006	0.782	#13	0.044	Not Required	Pass
307	0.018	0.442	0.393	0.071	0.079	0.510	#21	0.073	Not Required	Pass
308	0.004	0.071	0.234	0.048	0.020	0.299	#21	0.088	Not Required	Pass
309	0.028	0.047	0.071	0.001	0.000	0.124	#13	0.198	Not Required	Pass
310	0.017	0.726	0.363	0.073	0.060	0.894	#21	0.078	Not Required	Pass
311	0.006	0.053	0.241	0.046	0.019	0.297	#21	0.088	Not Required	Pass
312	0.006	0.438	0.291	0.107	0.052	0.689	#13	0.053	Not Required	Pass
313	0.008	0.235	0.513	0.057	0.024	0.720	#21	0.265	Not Required	Pass
314	0.009	0.259	0.504	0.059	0.024	0.726	#21	0.265	Not Required	Pass
315	0.031	0.292	0.275	0.045	0.019	0.550	#21	0.858	Not Required	Pass
316	0.009	0.282	0.273	0.045	0.019	0.528	#21	0.858	Not Required	Pass
401	0.052	0.663	0.061	0.047	0.006	0.706	#13	0.436	Not Required	Pass
402	0.004	0.425	0.261	0.104	0.049	0.654	#13	0.052	Not Required	Pass
403	0.018	0.661	0.176	0.068	0.029	0.778	#21	0.044	Not Required	Pass
404	0.019	0.637	0.363	0.064	0.061	0.782	#21	0.078	Not Required	Pass
405	0.019	0.411	0.392	0.066	0.078	0.473	#21	0.073	Not Required	Pass
406	0.012	0.488	0.069	0.048	0.009	0.501	#13	0.044	Not Required	Pass
407	0.011	0.303	0.203	0.049	0.041	0.325	#21	0.073	Not Required	Pass
408	0.000	0.054	0.129	0.022	0.010	0.178	#21	Not Required	Not Required	Pass
409	0.019	0.052	0.105	0.004	0.005	0.132	#13	0.198	Not Required	Pass
410	0.010	0.489	0.230	0.049	0.040	0.616	#21	0.078	Not Required	Pass
411	0.000	0.054	0.129	0.022	0.010	0.178	#21	Not Required	Not Required	Pass
412	0.006	0.270	0.190	0.068	0.038	0.425	#13	0.053	Not Required	Pass
413	0.008	0.167	0.511	0.053	0.024	0.643	#21	0.177	Not Required	Pass
414	0.008	0.160	0.500	0.052	0.024	0.611	#21	0.265	Not Required	Pass
415	0.031	0.368	0.279	0.043	0.019	0.620	#21	0.858	Not Required	Pass
416	0.009	0.369	0.276	0.041	0.019	0.597	#21	0.858	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
------------	--------------	---------

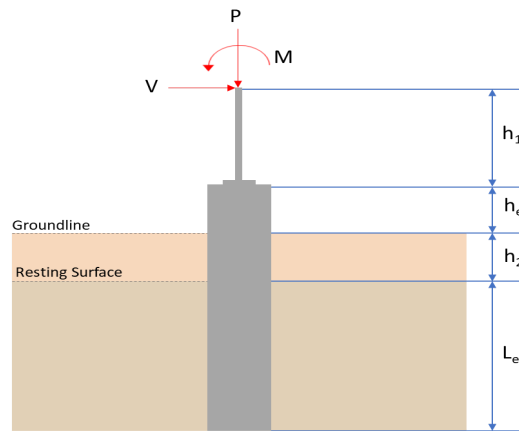
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.007	7.548
V_x (kip)	-2.140	-3.572
V_z (kip)	0.305	0.452
M_x (kipft)	0.632	0.933
M_z (kipft)	16.646	28.025

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.14 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.34076 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.646 \text{ kipft}) + ((-2.14 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6506 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8369 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.305 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.048567 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.632 \text{ kipft}) + ((0.305 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10064 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.4816 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8369 \text{ ft}), (2.4816 \text{ ft})]$$

$$L_{e,req} = 4.837 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.837 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.92133$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.007 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31294 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31294 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15647$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.34076 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6506 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6506 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.34076 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.6506 \text{ kipft/ft})) + (4 \times (-0.34076 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6358 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6506 \text{ kipft/ft})) + (3 \times (-0.34076 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.6506 \text{ kipft/ft})) + (2 \times (-0.34076 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.17053 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6506 \text{ kipft/ft})) + ((-0.34076 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.76457 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6358 \text{ ft})}{2}$$

$$p_a = 0.27268 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.17053 \text{ kip/ft}^2)}{(0.27268 \text{ kip/ft}^2)}$$

$$Ratio = 0.62537$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.76457 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.97089$$

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

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

Considering z-direction:

$H_o = 0.048567 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10064 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.048567 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.10064 \text{ kipft/ft})) + (4 \times (0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7748 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.10064 \text{ kipft/ft})) + (3 \times (0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.10064 \text{ kipft/ft})) + (2 \times (0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.045683 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10064 \text{ kipft/ft})) + ((0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.09932 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7748 \text{ ft})}{2}$$

$$p_a = 0.28311 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.045683 \text{ kip/ft}^2)}{(0.28311 \text{ kip/ft}^2)}$$

$$Ratio = 0.16136$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

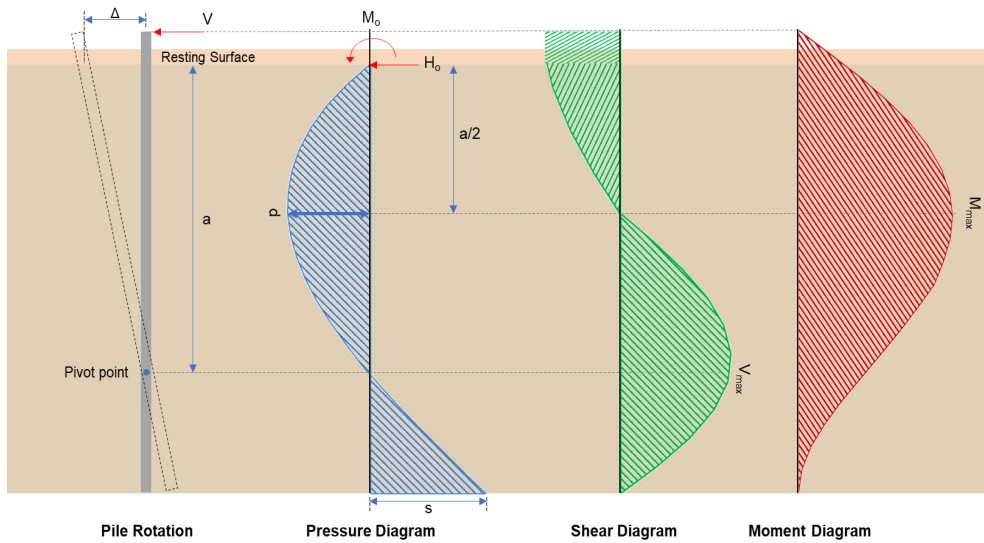
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.09932 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.12612$$

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

Status: **PASS**
Ratio: **0.130**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.572 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.56879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.025 \text{ kipft}) + ((-3.572 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.4626 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.4626 \text{ kipft/ft})}{(-0.56879 \text{ kip/ft})}$$

$$E = 7.8457 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4626 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.4626 \text{ kipft/ft})) + (4 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{6 \times (4.4626 \text{ kipft/ft}) + (4 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft}))}{a = 3.635 \text{ ft}}$$

$$a = 3.635 \text{ ft}$$

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

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

$$V_{max} = ((-0.56879 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.8457 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.635 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.8457 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.635 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.5166 \text{ kip}$$

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

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

$$M_{max} = ((-0.56879 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(7.8457 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.8457 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.8457 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.649 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.452 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.071975 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.933 \text{ kipft}) + ((0.452 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14857 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.14857 \text{ kipft/ft})}{(0.071975 \text{ kip/ft})}$$

$$E = 2.0642 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.14857 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.071975 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.14857 \text{ kipft/ft})) + (4 \times (0.071975 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7752 \text{ ft}$$

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

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

$$V_{max} = ((0.071975 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0642 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7752 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.0642 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7752 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39283 \text{ kip}$$

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

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

$$M_{max} = ((0.071975 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.0642 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7752 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.0642 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7752 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0642 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7752 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.89678 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.548 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.345 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.345 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(7.548 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0028215$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.548 \text{ kip} \rightarrow 7548 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7548 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.49 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.49 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.49 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.49 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.75 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.5166 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.5166 \text{ kip})}{(110.75 \text{ kip})}$$

$$Ratio = 0.067869$$

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

Considering z-direction:

$V_{max} = 0.39283 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.39283 \text{ kip})}{(110.75 \text{ kip})}$$

$$Ratio = 0.003547$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 18.649 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(18.649 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.074714$$

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

Considering z-direction:

$M_{max} = 0.89678 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.89678 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0035929$$

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

REFERENCES	CALCULATIONS	RESULTS
------------	--------------	---------

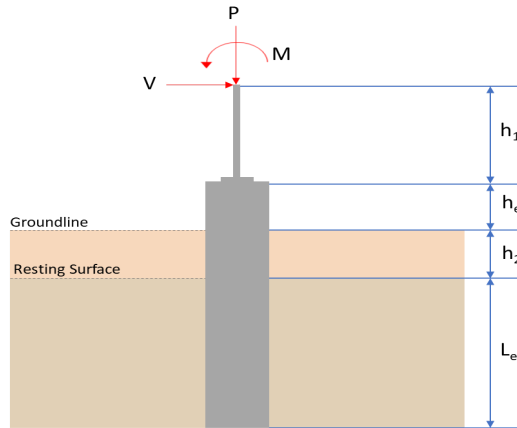
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.007	7.548
V_x (kip)	-2.139	-3.572
V_z (kip)	-0.305	-0.451
M_x (kipft)	-0.635	-0.937
M_z (kipft)	16.645	28.023

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.139 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.34061 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.645 \text{ kipft}) + ((-2.139 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6505 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8372 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.305 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.048567 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.635 \text{ kipft}) + ((-0.305 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10111 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5349 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8372 \text{ ft}), (1.5349 \text{ ft})]$$

$$L_{e,req} = 4.837 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.837 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.92133$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(5.007 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.31294 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.31294 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.15647$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.34061 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6505 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6505 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.34061 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.6505 \text{ kipft/ft})) + (4 \times (-0.34061 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6357 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6505 \text{ kipft/ft})) + (3 \times (-0.34061 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (2.6505 \text{ kipft/ft})) + (2 \times (-0.34061 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.1706 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6505 \text{ kipft/ft})) + ((-0.34061 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.76469 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6357 \text{ ft})}{2}$$

$$p_a = 0.27268 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.1706 \text{ kip/ft}^2)}{(0.27268 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.62565$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.76469 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97103$$

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

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

Considering z-direction:

$H_o = -0.048567 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.10111 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10111 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.048567 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.10111 \text{ kipft/ft})) + (4 \times (-0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7743 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.10111 \text{ kipft/ft})) + (3 \times (-0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.10111 \text{ kipft/ft})) + (2 \times (-0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = -0.017113 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.10111 \text{ kipft/ft})) + ((-0.048567 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = -0.011482 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7743 \text{ ft})}{2}$$

$$p_a = 0.28307 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.017113 \text{ kip/ft}^2)}{(0.28307 \text{ kip/ft}^2)}$$

$$Ratio = -0.060455$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$$

$$p_s = 0.7875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

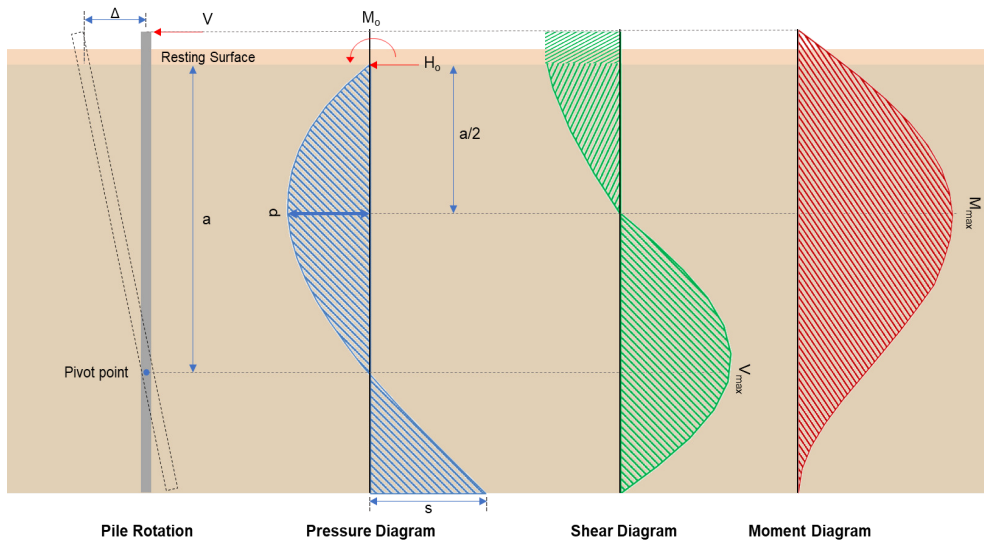
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.011482 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = -0.014581$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.572 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.56879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(28.023 \text{ kipft}) + ((-3.572 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.4623 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.4623 \text{ kipft/ft})}{(-0.56879 \text{ kip/ft})}$$

$$E = 7.8452 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.4623 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (4.4623 \text{ kipft/ft})) + (4 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = \frac{(-0.56879 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.4623 \text{ kipft/ft})) + (4 \times (-0.56879 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.635 \text{ ft}$$

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

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

$$V_{max} = ((-0.56879 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.8452 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.635 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.8452 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.635 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.5161 \text{ kip}$$

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

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

$$M_{max} = ((-0.56879 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(7.8452 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.8452 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.8452 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.647 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.451 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.071815 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.937 \text{ kipft}) + ((-0.451 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.1492 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.1492 \text{ kipft/ft})}{(-0.071815 \text{ kip/ft})}$$

$$E = 2.0776 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.1492 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.071815 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.1492 \text{ kipft/ft})) + (4 \times (-0.071815 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7745 \text{ ft}$$

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

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

$$V_{max} = ((-0.071815 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.0776 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7745 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.0776 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7745 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39324 \text{ kip}$$

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

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

$$M_{max} = ((-0.071815 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(2.0776 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7745 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.0776 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7745 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.0776 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7745 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.89815 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.548 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.345 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.345 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.548 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0028215$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.548 \text{ kip} \rightarrow 7548 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7548 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.49 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.49 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.49 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.49 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.75 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.5161 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.5161 \text{ kip})}{(110.75 \text{ kip})}$$

$$Ratio = 0.067865$$

Considering z-direction:

$V_{max} = 0.39324 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.39324 \text{ kip})}{(110.75 \text{ kip})}$$

$$Ratio = 0.0035507$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 18.647 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.647 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.074709$$

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

Considering z-direction:

$M_{max} = 0.89815 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.89815 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0035984$$

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

REFERENCES	CALCULATIONS	RESULTS
------------	--------------	---------

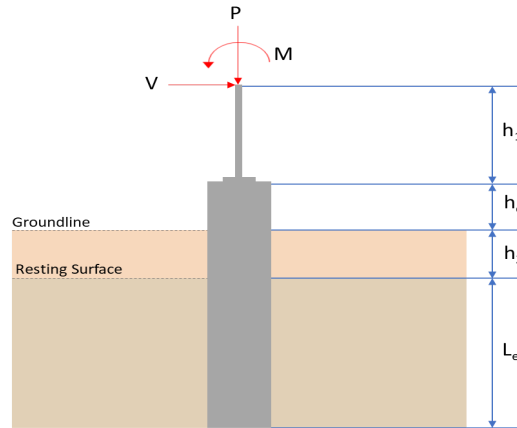
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.116	9.222
V_x (kip)	-2.657	-4.433
V_z (kip)	-0.029	-0.044
M_x (kipft)	-0.067	-0.101
M_z (kipft)	20.770	35.255

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.657 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42309 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.77 \text{ kipft}) + ((-2.657 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3073 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1238 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.029 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.067 \text{ kipft}) + ((-0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010669 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.85157 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1238 \text{ ft}), (0.85157 \text{ ft})]$$

$$L_{e,req} = 5.124 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.124 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.89113$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.116 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38225 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38225 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19112$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42309 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3073 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3073 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.3073 \text{ kipft/ft})) + (4 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.991 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3073 \text{ kipft/ft})) + (3 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3073 \text{ kipft/ft})) + (2 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.15781 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3073 \text{ kipft/ft})) + ((-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.75891 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.991 \text{ ft})}{2}$$

$$p_a = 0.29932 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.15781 \text{ kip/ft}^2)}{(0.29932 \text{ kip/ft}^2)}$$

$$Ratio = 0.52722$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.75891 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.87989$$

Status: **PASS**
Ratio: **0.530**

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = -0.0046178 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010669 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010669 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.010669 \text{ kipft/ft})) + (4 \times (-0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1323 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.010669 \text{ kipft/ft})) + (3 \times (-0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.010669 \text{ kipft/ft})) + (2 \times (-0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = -0.0014705 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010669 \text{ kipft/ft})) + ((-0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = -0.00094638 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1323 \text{ ft})}{2}$$

$$p_a = 0.30992 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0014705 \text{ kip/ft}^2)}{(0.30992 \text{ kip/ft}^2)}$$

$$Ratio = -0.0047447$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

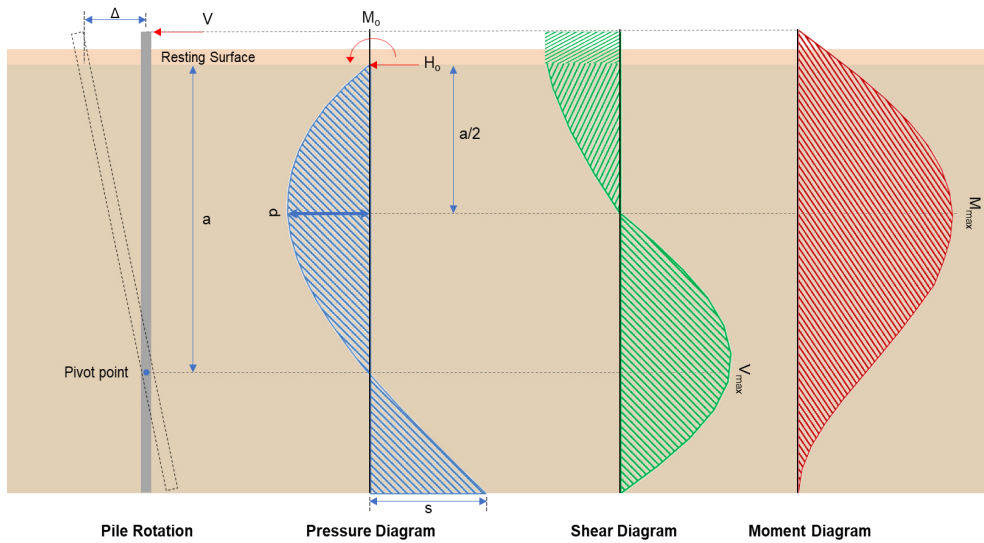
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00094638 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.0010973$$

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,

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

$$H_o = \frac{(-4.433 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70589 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.255 \text{ kipft}) + ((-4.433 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6139 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6139 \text{ kipft/ft})}{(-0.70589 \text{ kip/ft})}$$

$$E = 7.9529 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6139 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.6139 \text{ kipft/ft})) + (4 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.70589 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.6139 \text{ kipft/ft})) + (4 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9892 \text{ ft}$$

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

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

$$V_{max} = ((-0.70589 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9892 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9892 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.7722 \text{ kip}$$

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

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

$$M_{max} = ((-0.70589 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(7.9529 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.751 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.044 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0070064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.101 \text{ kipft}) + ((-0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.016083 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.016083 \text{ kipft/ft})}{(-0.0070064 \text{ kip/ft})}$$

$$E = 2.2955 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.016083 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.0070064 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.016083 \text{ kipft/ft})) + (4 \times (-0.0070064 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.133 \text{ ft}$$

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

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

$$V_{max} = ((-0.0070064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.2955 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.133 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.2955 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.133 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.038535 \text{ kip}$$

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

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

$$M_{max} = ((-0.0070064 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.2955 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.133 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.2955 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.133 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.2955 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.133 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.096456 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.222 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.29 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.29 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(9.222 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0034472$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.222 \text{ kip} \rightarrow 9222 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9222 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.71 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.9 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.7722 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.7722 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.079104$$

Considering z-direction:

$V_{max} = 0.038535 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.038535 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.00034749$$

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

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:

$M_{max} = 23.751 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.751 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.095155$$

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

Considering z-direction:

$M_{max} = 0.096456 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.096456 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00038644$$

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

REFERENCES	CALCULATIONS	RESULTS
------------	--------------	---------

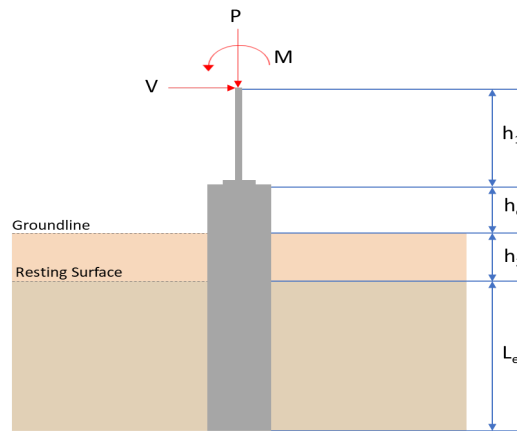
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.024	9.090
V_x (kip)	-2.591	-4.320
V_z (kip)	0.000	0.000
M_x (kipft)	-0.002	-0.003
M_z (kipft)	20.433	34.658

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.591 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.41258 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.433 \text{ kipft}) + ((-2.591 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2537 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1135 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.002 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00031847 \text{ kipft/ft}$$

L_e - Required depth of embedment in earth,

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

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.00031847 \text{ kipft/ft})}{(150 \text{ psf/ft})}}$$

$$L_e = 0.29433 \text{ ft}$$

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1135 \text{ ft}), (0.29433 \text{ ft})]$$

$$L_{e,req} = 5.114 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.114 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.88939$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.024 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.3765 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.3765 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18825$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.41258 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.2537 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2537 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.41258 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.2537 \text{ kipft/ft})) + (4 \times (-0.41258 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9901 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2537 \text{ kipft/ft})) + (3 \times (-0.41258 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.2537 \text{ kipft/ft})) + (2 \times (-0.41258 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.15729 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.2537 \text{ kipft/ft})) + ((-0.41258 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.7504 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9901 \text{ ft})}{2}$$

$$p_a = 0.29925 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.15729 \text{ kip/ft}^2)}{(0.29925 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.5256$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.7504 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.87002$$

Status: **PASS**
Ratio: **0.530**

Status: **PASS**
Ratio: **0.870**

Considering z-direction:

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

$M_o = 0.00031847 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00031847 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.00031847 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8333 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.00031847 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.00031847 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.00003853 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.00031847 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.00011559 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8333 \text{ ft})}{2}$$

$$p_a = 0.2875 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0003853 \text{ kip/ft}^2)}{(0.2875 \text{ kip/ft}^2)}$$

$$Ratio = 0.00013402$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

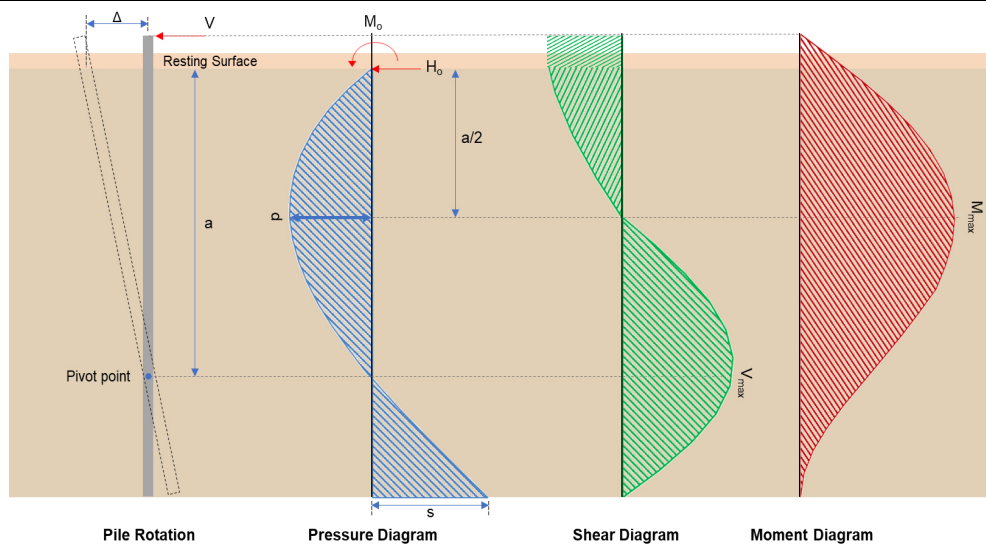
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00011559 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.00013402$$

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,

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

$$H_o = \frac{(-4.32 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.6879 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(34.658 \text{ kipft}) + ((-4.32 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5188 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5188 \text{ kipft/ft})}{(-0.6879 \text{ kip/ft})}$$

$$E = 8.0227 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

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

$$a = \frac{(4 \times (5.5188 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.6879 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.5188 \text{ kipft/ft})) + (4 \times (-0.6879 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9883 \text{ ft}$$

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

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

$$V_{max} = ((-0.6879 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (8.0227 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9883 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (8.0227 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9883 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6077 \text{ kip}$$

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

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

$$M_{max} = ((-0.6879 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(8.0227 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9883 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (8.0227 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9883 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (8.0227 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9883 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.315 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.003 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00047771 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00047771 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.00047771 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8333 \text{ ft}$$

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

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

$$V_{max} = 12 \times \left(\frac{(0.00047771 \text{ kipft/ft}) \times (48 \text{ in})}{(5.75 \text{ ft})} \right) \times \left(\frac{(3.8333 \text{ ft})}{(5.75 \text{ ft})} - 1 \right) \times \left(\frac{(3.8333 \text{ ft})}{(5.75 \text{ ft})} \right)^2$$

$$V_{max} = 0.00059079 \text{ kip}$$

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

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

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

$$M_{max} = ((0.00047771 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(3.8333 \text{ ft})^3}{2 \times (5.75 \text{ ft})} \right) + \left(3 \times \frac{(3.8333 \text{ ft})^4}{2 \times (5.75 \text{ ft})} \right) \right]$$

$$M_{max} = 0.0016985 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.09 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.294 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.294 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

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

Summary:

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

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

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(9.09 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0033979$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.09 \text{ kip} \rightarrow 9090 \text{ lbf}$,

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(9090 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

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

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

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

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

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

V_c - Governing shear strength of concrete

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

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

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

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

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

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

$$\phi V_n = 110.88 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.6077 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.6077 \text{ kip})}{(110.88 \text{ kip})}$$

$$\text{Ratio} = 0.077628$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$$\phi M_{n,1}$$

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

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

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

14.5.2.1b

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

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 23.315 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(23.315 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.093409$$

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

Considering z-direction:

$M_{max} = 0.0016985 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.0016985 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 6.8049 \times 10^{-6}$$

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

REFERENCES	CALCULATIONS	RESULTS
------------	--------------	---------

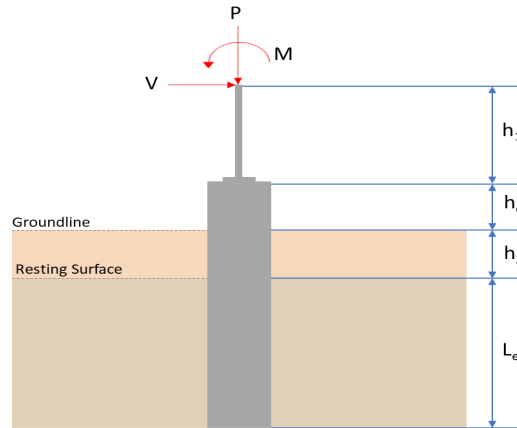
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.116	9.222
V_x (kip)	-2.657	-4.433
V_z (kip)	0.029	0.044
M_x (kipft)	0.063	0.095
M_z (kipft)	20.770	35.255

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.657 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42309 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.77 \text{ kipft}) + ((-2.657 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3073 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.1238 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.029 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.063 \text{ kipft}) + ((0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010032 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0283 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1238 \text{ ft}), (1.0283 \text{ ft})]$$

$$L_{e,req} = 5.124 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.124 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.89113$$

Status: **PASS**
Ratio: **0.890**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.116 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38225 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38225 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19112$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.42309 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3073 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3073 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (3.3073 \text{ kipft/ft})) + (4 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.991 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3073 \text{ kipft/ft})) + (3 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (3.3073 \text{ kipft/ft})) + (2 \times (-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.15781 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3073 \text{ kipft/ft})) + ((-0.42309 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.75891 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.991 \text{ ft})}{2}$$

$$p_a = 0.29932 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.15781 \text{ kip/ft}^2)}{(0.29932 \text{ kip/ft}^2)}$$

$$Ratio = 0.52722$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.75891 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.87989$$

Status: **PASS**
Ratio: **0.530**

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = 0.0046178 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.010032 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010032 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.010032 \text{ kipft/ft})) + (4 \times (0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1392 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.010032 \text{ kipft/ft})) + (3 \times (0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.010032 \text{ kipft/ft})) + (2 \times (0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.003912 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.010032 \text{ kipft/ft})) + ((0.0046178 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.0084597 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1392 \text{ ft})}{2}$$

$$p_a = 0.31044 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.003912 \text{ kip/ft}^2)}{(0.31044 \text{ kip/ft}^2)}$$

$$Ratio = 0.012602$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

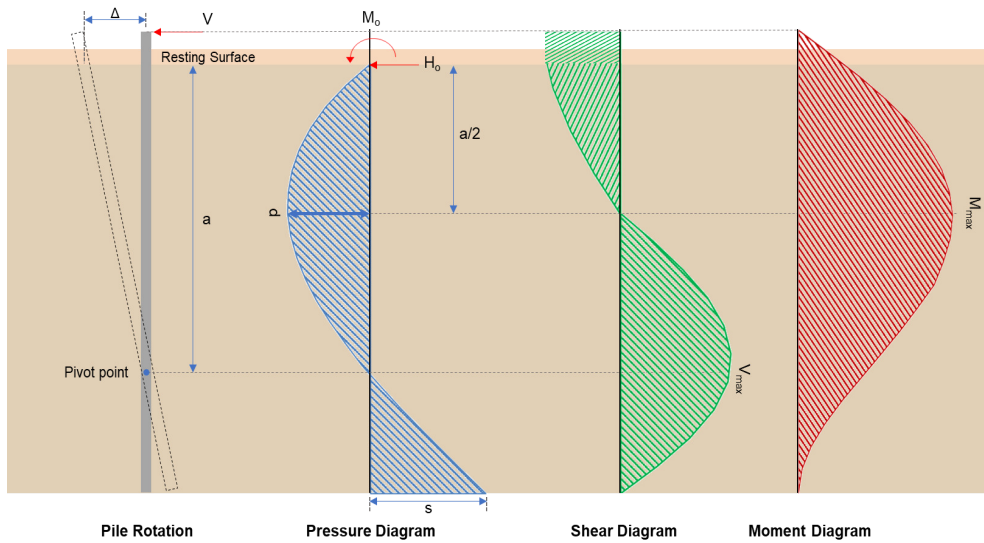
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0084597 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0098083$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.433 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.70589 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.255 \text{ kipft}) + ((-4.433 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6139 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6139 \text{ kipft/ft})}{(-0.70589 \text{ kip/ft})}$$

$$E = 7.9529 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6139 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (5.6139 \text{ kipft/ft})) + (4 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.70589 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.6139 \text{ kipft/ft})) + (4 \times (-0.70589 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9892 \text{ ft}$$

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

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

$$V_{max} = ((-0.70589 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9892 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9892 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.7722 \text{ kip}$$

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

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

$$M_{max} = ((-0.70589 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(7.9529 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.9529 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9892 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 23.751 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.044 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0070064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.095 \text{ kipft}) + ((0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.015127 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.015127 \text{ kipft/ft})}{(0.0070064 \text{ kip/ft})}$$

$$E = 2.1591 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.015127 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.0070064 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.015127 \text{ kipft/ft})) + (4 \times (0.0070064 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1399 \text{ ft}$$

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

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

$$V_{max} = ((0.0070064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.1591 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1399 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.1591 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1399 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.037376 \text{ kip}$$

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

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

$$M_{max} = ((0.0070064 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.1591 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1399 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.1591 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1399 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.1591 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1399 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.093137 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(9.222 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.29 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.29 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(9.222 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0034472$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 9.222 \text{ kip} \rightarrow 9222 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(9222 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.71 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.71 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.71 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.71 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.9 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.7722 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(8.7722 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.079104$$

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

Considering z-direction:

$V_{max} = 0.037376 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.037376 \text{ kip})}{(110.9 \text{ kip})}$$

$$Ratio = 0.00033704$$

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:

$M_{max} = 23.751 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(23.751 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.095155$$

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

Considering z-direction:

$M_{max} = 0.093137 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.093137 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00037315$$

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