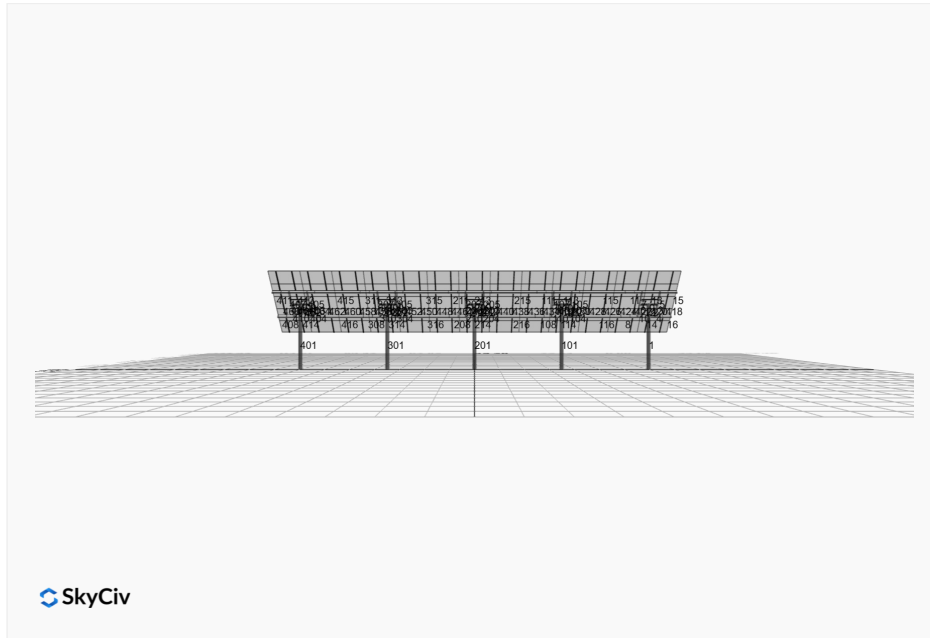


Project Details



Project Name: BT SIL530 46d 8h II **Date:** Mon Jul 07 2025
Location: 31842 Frontage Rd, Bozeman, MT 59715, USA **Number of Modules:** 65
Unique ID: 5P-19.75-8TOP-HD-24-L-5Hx13W-98A1 **Number of Poles:** 5
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	90.57 ft
Winter Tilt Angle	46
Front Edge Clearance	8 ft

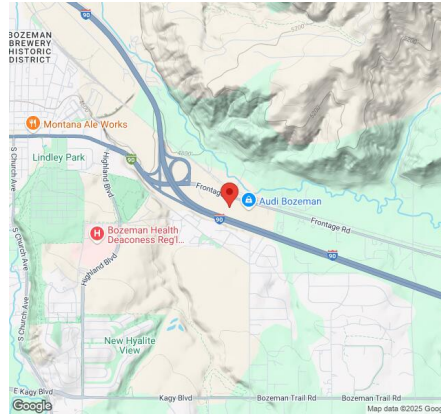
MT Solar Bill of Materials (5P-19.75-8TOP-HD-24-L-5Hx13W-98A1)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	13

Rail Bill of Materials

Part	Qty
Rails (226in)	26
Rail Attachment	104
Module Mid Clamp	104
Module End Clamp	52
Ground Lug	13

Site Details:



Site Address: 31842 Frontage Rd, Bozeman, MT 59715, USA

Array Specification

Duty Classification:	HD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Winter Tilt Angle:	46
Front Edge Clearance:	8
Total Array Height at Tilt:	21.52 ft
Total Frame Length:	90.50 ft
Module Info/Notes:	sil-530-xm
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	90.57 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	8in Pipe Sch 80
Pole Length above Grade:	14.76 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 7.50 ft Pile 2: 8.00 ft Pile 3: 8.25 ft Pile 4: 8.00 ft Pile 5: 7.50 ft
Foundation Volume:	23.259 y ³

Site Info

Risk Category:	II
Exposure:	C
Soil Classification:	sand
Site Location:	31842 Frontage Rd, Bozeman, MT 59715, USA
Wind Speed:	115 mph

Snow Load:

51 psf

Design Disclaimer

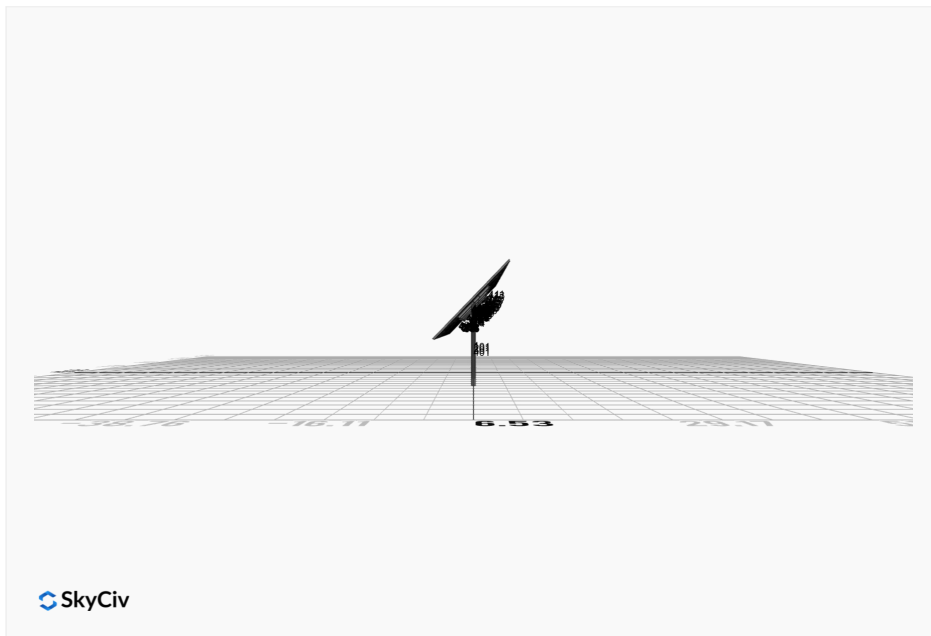
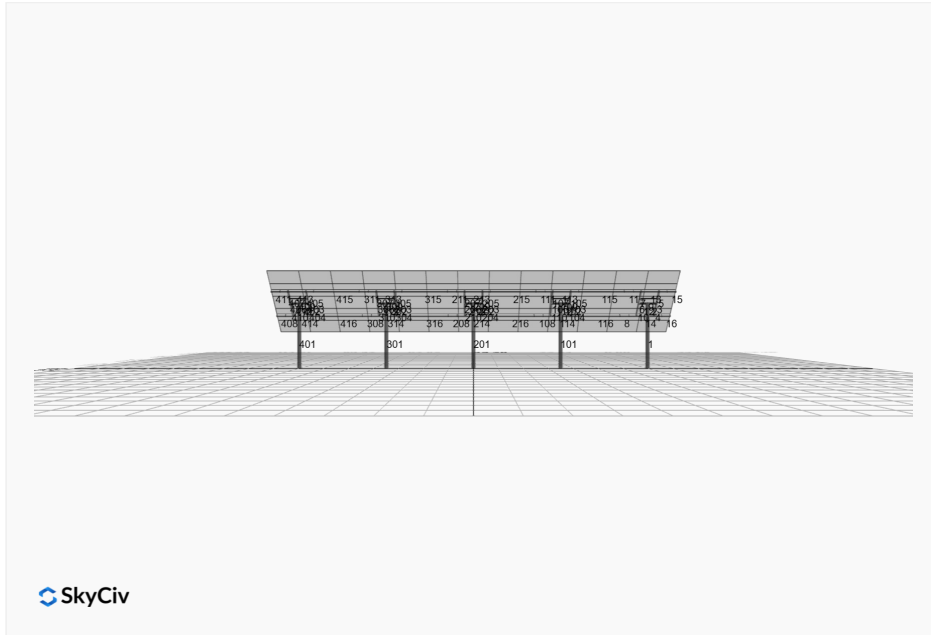
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

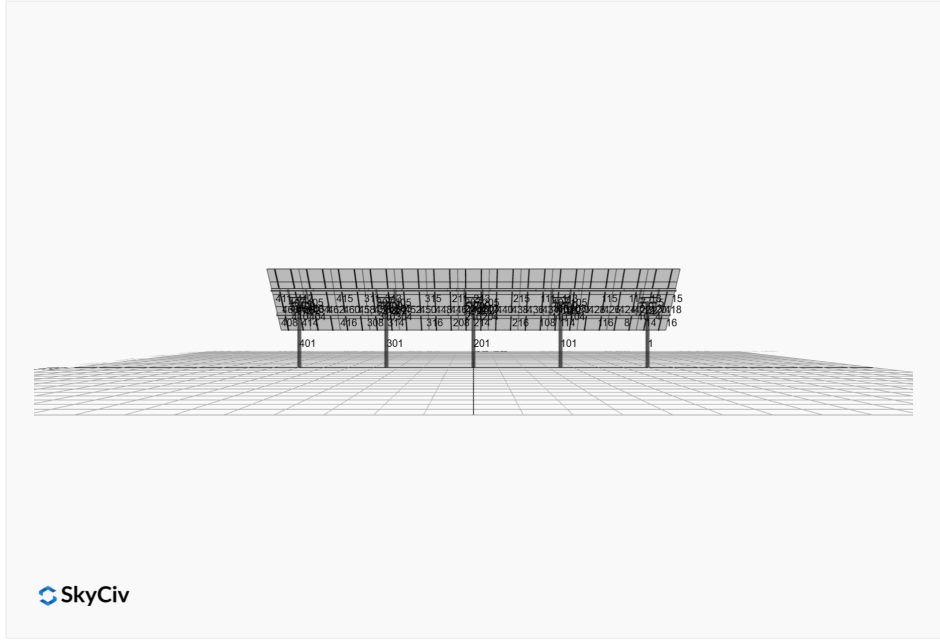
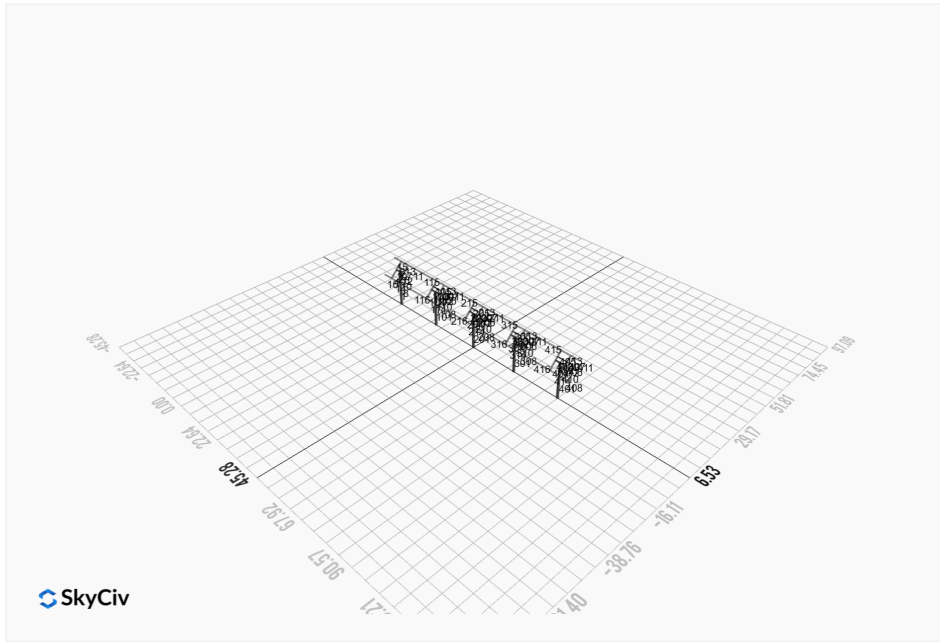
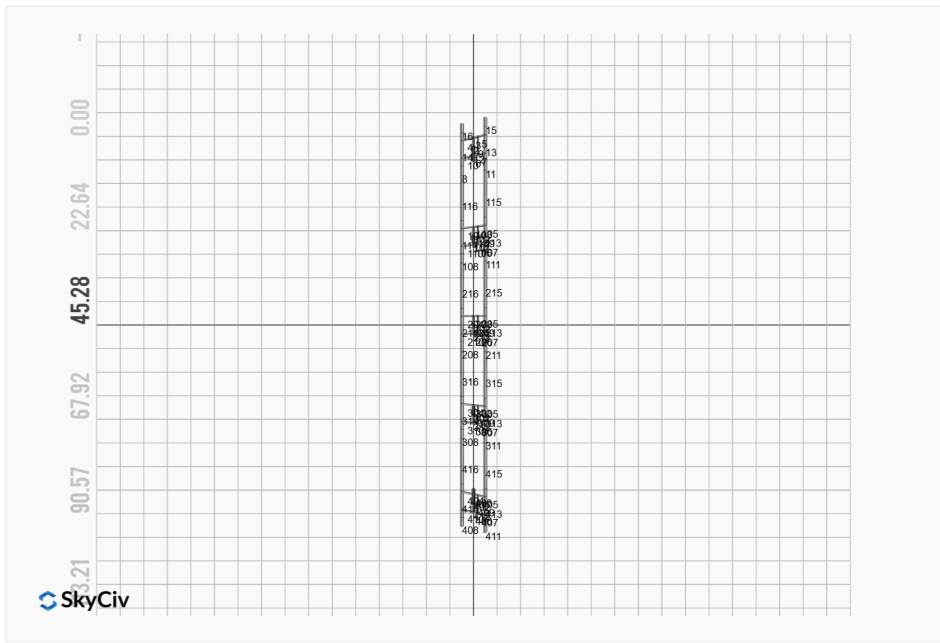
AutoDesigner Input

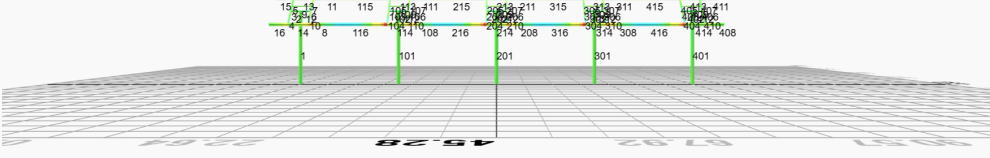
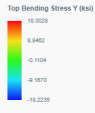
```
{ "wind_speed_override": 115, "snow_load_override": 51, "direct_snow_load": false, "add_angle_brace": false, "product_type": "Beam", "designer_name": "Thomas Burney", "designer_email": "thomas.burney@onsiteenergyinc.com", "designer_phone": "", "project_id": "BT SIL530 46d 8h II", "site_address": "31842 Frontage Rd, Bozeman, MT 59715, USA", "module_info": "sil-530-xm", "module_width": 44.6, "module_length": 82.6, "number_rows": 5, "number_columns": 13, "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_12GA", "pole_spacing": 15, "tilt_angle": 46, "ground_clearance": 8, "risk_category": "II", "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:

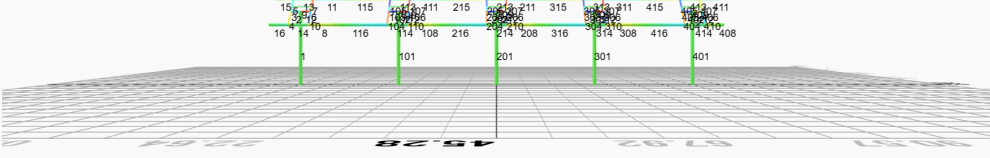
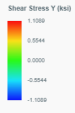
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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







SkyCiv



SkyCiv

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 2. D + L	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 3. D + (S or Lr or R)	0.0786	5.8176	0.2095	0.9584	-0.3099	-1.0739
ULS: 3. D + (S or Lr or R)	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0653	4.9753	0.1739	0.7955	-0.2572	-0.8888
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 5b. D + 0.7E	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0653	4.9753	0.1739	0.7955	-0.2572	-0.8888
ULS: 8. 0.6D + 0.7E	0.0152	1.4689	0.0403	0.1840	-0.0594	-0.2002
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8319	5.1345	0.2907	1.2826	-1.3989	42.7629
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8752	-0.2345	-0.1497	-0.6376	1.1619	-42.0019
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0776	6.9900	0.3416	1.5274	-1.2321	31.4336
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0653	4.9753	0.1739	0.7955	-0.2572	-0.8888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2027	2.9633	0.0112	0.0873	0.6885	-32.1401
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0653	4.9753	0.1739	0.7955	-0.2572	-0.8888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1176	4.4629	0.2348	1.0386	-1.0739	31.9888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1627	0.4362	-0.0955	-0.4015	0.8467	-31.5849
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0253	2.4481	0.0671	0.3067	-0.0990	-0.3336
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8420	4.1552	0.2639	1.1599	-1.3593	42.8963
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0152	1.4689	0.0403	0.1840	-0.0594	-0.2002
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8651	-1.2137	-0.1766	-0.7603	1.2015	-41.8685
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0152	1.4689	0.0403	0.1840	-0.0594	-0.2002

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	10.5692
Shear X	-4.8121
Shear Z	0.5296
Moment X	2.3454
Moment Y (Twist)	2.4201
Moment Z	72.3908

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	6.9900
Shear X	-2.8752
Shear Z	0.3416
Moment X	1.5274
Moment Y (Twist)	1.3989
Moment Z	42.8963

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	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 2. D + L	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 3. D + (S or Lr or R)	-0.0709	7.3316	-0.0104	-0.0503	0.0596	1.0079
ULS: 3. D + (S or Lr or R)	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0589	6.2323	-0.0086	-0.0418	0.0495	0.8387

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 5b. D + 0.7E	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0589	6.2323	-0.0086	-0.0418	0.0495	0.8387
ULS: 8. 0.6D + 0.7E	-0.0137	1.7608	-0.0020	-0.0098	0.0116	0.1985
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6170	6.4698	0.0283	0.1157	-0.2440	54.1549
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5787	-0.6054	-0.0304	-0.1281	0.2523	-51.5027
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7546	8.8837	0.0152	0.0572	-0.1479	41.2067
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0589	6.2323	-0.0086	-0.0418	0.0495	0.8387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6422	3.5773	-0.0289	-0.1257	0.2243	-38.0365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0589	6.2323	-0.0086	-0.0418	0.0495	0.8387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7185	5.5860	0.0204	0.0827	-0.1782	40.6989
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6783	0.2796	-0.0236	-0.1001	0.1940	-38.5443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0228	2.9346	-0.0033	-0.0163	0.0193	0.3308
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6079	5.2960	0.0297	0.1222	-0.2517	54.0226
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0137	1.7608	-0.0020	-0.0098	0.0116	0.1985
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5878	-1.7792	-0.0291	-0.1216	0.2446	-51.6350
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0137	1.7608	-0.0020	-0.0098	0.0116	0.1985

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	13.4994
Shear X	-6.0352
Shear Z	-0.0558
Moment X	-0.2363
Moment Y (Twist)	0.4532
Moment Z	92.2789

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.8837
Shear X	-3.6170
Shear Z	-0.0304
Moment X	-0.1281
Moment Y (Twist)	0.2523
Moment Z	54.1549

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	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 2. D + L	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 3. D + (S or Lr or R)	-0.0154	7.2518	0.0000	-0.0001	0.0001	0.3606
ULS: 3. D + (S or Lr or R)	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0128	6.1661	0.0000	-0.0001	0.0001	0.3011
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 5b. D + 0.7E	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0128	6.1661	0.0000	-0.0001	0.0001	0.3011
ULS: 8. 0.6D + 0.7E	-0.0029	1.7454	0.0000	-0.0000	0.0000	0.0735
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6446	6.4408	0.0000	-0.0000	0.0000	55.0268
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.6348	-0.6209	0.0000	-0.0000	0.0000	-52.6239
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7426	8.8150	0.0000	-0.0001	0.0001	41.4793
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0128	6.1661	0.0000	-0.0001	0.0001	0.3011
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7170	3.5188	0.0000	-0.0001	0.0001	-39.2587
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0128	6.1661	0.0000	-0.0001	0.0001	0.3011
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7347	5.5578	0.0000	-0.0000	0.0000	41.3007
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7249	0.2616	0.0000	-0.0000	0.0000	-39.4373
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0049	2.9089	0.0000	-0.0000	0.0000	0.1225
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6427	5.2772	0.0000	-0.0000	0.0000	54.9778
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0029	1.7454	0.0000	-0.0000	0.0000	0.0735
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.6368	-1.7844	0.0000	-0.0000	0.0000	-52.6729
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0029	1.7454	0.0000	-0.0000	0.0000	0.0735

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	13.3857
Shear X	-6.0781
Shear Z	0.0000
Moment X	-0.0006
Moment Y (Twist)	0.0006
Moment Z	93.7551

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.8150
Shear X	-3.6446
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0001
Moment Z	55.0268

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	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 2. D + L	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 3. D + (S or Lr or R)	-0.0709	7.3316	0.0104	0.0502	-0.0594	1.0079
ULS: 3. D + (S or Lr or R)	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0589	6.2323	0.0086	0.0417	-0.0494	0.8387
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 5b. D + 0.7E	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0589	6.2323	0.0086	0.0417	-0.0494	0.8387
ULS: 8. 0.6D + 0.7E	-0.0137	1.7608	0.0020	0.0097	-0.0115	0.1985
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.6170	6.4698	-0.0283	-0.1158	0.2440	54.1549
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.5787	-0.6054	0.0304	0.1281	-0.2522	-51.5027
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7546	8.8837	-0.0151	-0.0573	0.1480	41.2067
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0589	6.2323	0.0086	0.0417	-0.0494	0.8387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6422	3.5773	0.0289	0.1256	-0.2241	-38.0365
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0589	6.2323	0.0086	0.0417	-0.0494	0.8387
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.7185	5.5860	-0.0204	-0.0828	0.1782	40.6989
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.6783	0.2796	0.0236	0.1001	-0.1940	-38.5443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0228	2.9346	0.0033	0.0162	-0.0192	0.3308

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.6079	5.2960	-0.0297	-0.1223	0.2517	54.0226
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0137	1.7608	0.0020	0.0097	-0.0115	0.1985
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.5878	-1.7792	0.0291	0.1216	-0.2445	-51.6350
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0137	1.7608	0.0020	0.0097	-0.0115	0.1985

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	13.4995
Shear X	-6.0352
Shear Z	0.0558
Moment X	0.2368
Moment Y (Twist)	0.4532
Moment Z	92.2792

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	8.8837
Shear X	-3.6170
Shear Z	0.0304
Moment X	0.1281
Moment Y (Twist)	0.2522
Moment Z	54.1549

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	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 2. D + L	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 3. D + (S or Lr or R)	0.0786	5.8176	-0.2095	-0.9587	0.3101	-1.0737
ULS: 3. D + (S or Lr or R)	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0653	4.9752	-0.1739	-0.7957	0.2573	-0.8887
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 5b. D + 0.7E	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0653	4.9752	-0.1739	-0.7957	0.2573	-0.8887
ULS: 8. 0.6D + 0.7E	0.0152	1.4689	-0.0403	-0.1840	0.0594	-0.2002
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8319	5.1345	-0.2907	-1.2826	1.3989	42.7629
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8752	-0.2345	0.1497	0.6376	-1.1618	-42.0019
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0776	6.9900	-0.3416	-1.5276	1.2322	31.4337
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0653	4.9752	-0.1739	-0.7957	0.2573	-0.8887
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2027	2.9633	-0.0112	-0.0875	-0.6883	-32.1400
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0653	4.9752	-0.1739	-0.7957	0.2573	-0.8887
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1176	4.4629	-0.2348	-1.0386	1.0739	31.9888
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1627	0.4362	0.0955	0.4015	-0.8466	-31.5848
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0253	2.4481	-0.0671	-0.3067	0.0990	-0.3336
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8420	4.1552	-0.2639	-1.1599	1.3593	42.8964
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0152	1.4689	-0.0403	-0.1840	0.0594	-0.2002
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8651	-1.2137	0.1766	0.7603	-1.2014	-41.8685
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0152	1.4689	-0.0403	-0.1840	0.0594	-0.2002

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	10.5692
Shear X	-4.8121
Shear Z	-0.5296
Moment X	-2.3461
Moment Y (Twist)	2.4206
Moment Z	72.3921

Result	Value (kip, kip-ft)
Axial	6.9900
Shear X	-2.8752
Shear Z	-0.3416
Moment X	-1.5276
Moment Y (Twist)	1.3989
Moment Z	42.8964

Project Details

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

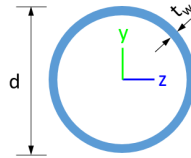


Design Input Information

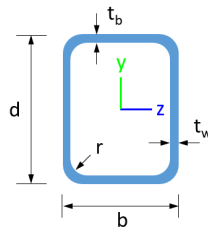
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

Design Materials			
ID	E (ksi)	F_y (ksi)	F_u (ksi)
1	29000	50	65

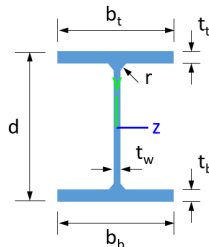
Section Dimensions



ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
10	8in Pipe Sch 80	8.63	0.50				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------

113	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.09,1.06,1.15,1.06,1.10,1.06,1.12,1.06,1.08,1.06,1.05,1.06,1.09,1.06,1.16,1.06,1.10,1.06,1.11,1.06	300	200	1
114	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.07,1.05,1.10,1.05,1.08,1.05,1.09,1.05,1.06,1.05,1.05,1.05,1.07,1.05,1.10,1.05,1.08,1.05,1.08,1.05	300	200	1
115	19	6.63	6.63	10.20	1.14,1.14,1.14,1.14,1.14,1.14,1.11,1.14,1.08,1.14,1.11,1.14,1.09,1.14,1.12,1.14,1.28,1.14,1.11,1.14,1.08,1.14,1.10,1.14,1.10,1.14,1.14	300	200	1
116	19	6.63	6.63	10.20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.13,1.15,1.15,1.15,1.18,1.15,1.14,1.15,1.13,1.15,1.14,1.15,1.14,1.15	300	200	1
201	10	30.99	30.99	14.76	-	300	200	1
202	5	1.30	1.30	2.00	-	300	200	1
203	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
204	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
205	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
206	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
207	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
208	19	1.33	1.33	2.05	2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.08,2.09,2.09,2.09,2.08,2.09,2.09,2.09,2.11,2.09,2.09,2.09,2.08,2.09,2.09,2.09,2.09	300	200	1
209	2	2.60	2.60	4.00	-	300	200	1
210	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.66,1.68	300	200	1
211	19	1.33	1.33	2.05	2.11,2.11,2.11,2.11,2.11,2.11,2.07,2.11,1.83,2.11,2.06,2.11,1.98,2.11,2.08,2.11,2.19,2.11,2.07,2.11,1.83,2.11,2.06,2.11,2.01,2.11	300	200	1
212	5	1.30	1.30	2.00	-	300	200	1
213	19	4.88	4.00	7.50	1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.03,1.04,1.04,1.04,1.04,1.04,1.04,1.04,1.04	300	200	1
214	19	4.88	4.00	7.50	1.04,1.04	300	200	1
215	19	6.63	6.63	10.20	1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.17,1.14,1.17,1.15,1.17,1.14,1.17,1.16,1.17,1.23,1.17,1.15,1.17,1.14,1.17,1.15,1.17,1.14,1.17	300	200	1
216	19	6.63	6.63	10.20	1.16,1.16	300	200	1
301	10	30.99	30.99	14.76	-	300	200	1
302	5	1.30	1.30	2.00	-	300	200	1
303	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	300	200	1
304	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.66,1.68	300	200	1
305	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
306	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.17,1.19	300	200	1
307	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68	300	200	1
308	19	1.33	1.33	2.05	2.02,2.03,2.02,2.03,2.02,2.02,1.80,2.03,1.65,2.03,1.78,2.02,1.71,2.02,1.89,2.03,2.14,2.03,1.81,2.02,1.64,2.02,1.77,2.02,1.73,2.02	300	200	1
309	2	2.60	2.60	4.00	-	300	200	1
310	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.66,1.68	300	200	1
311	19	1.33	1.33	2.05	1.83,1.83,1.83,1.82,1.83,1.83,1.49,1.83,1.32,1.83,1.46,1.83,1.39,1.83,1.60,1.82,2.22,1.82,1.50,1.83,1.33,1.83,1.45,1.83,1.40,1.83	300	200	1
312	5	1.30	1.30	2.00	-	300	200	1
313	19	4.88	4.00	7.50	1.06,1.06,1.06,1.06,1.06,1.06,1.09,1.06,1.15,1.06,1.10,1.06,1.12,1.06,1.08,1.06,1.05,1.06,1.09,1.06,1.16,1.06,1.10,1.06,1.11,1.06	300	200	1
314	19	4.88	4.00	7.50	1.05,1.05,1.05,1.05,1.05,1.05,1.07,1.05,1.10,1.05,1.08,1.05,1.09,1.05,1.06,1.05,1.05,1.05,1.07,1.05,1.10,1.05,1.08,1.05,1.08,1.05	300	200	1

104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.97	6.12	40.24	43.62
114	133.20	85.85	24.06	6.12	40.24	43.62
115	133.20	69.16	16.70	6.12	40.24	43.62
116	133.20	69.16	17.47	6.12	40.24	43.62
201	574.32	172.65	123.94	123.94	172.30	172.30
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.62	6.12	40.24	43.62
214	133.20	85.85	23.73	6.12	40.24	43.62
215	133.20	69.16	17.58	6.12	40.24	43.62
216	133.20	69.16	17.90	6.12	40.24	43.62
301	574.32	172.65	123.94	123.94	172.30	172.30
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	123.95	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	123.95	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	23.98	6.12	40.24	43.62
314	133.20	85.85	24.05	6.12	40.24	43.62
315	133.20	69.16	17.17	6.12	40.24	43.62
316	133.20	69.16	17.61	6.12	40.24	43.62
401	574.32	172.65	123.94	123.94	172.30	172.30
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28

407	110.10	114.23	13.79	11.10	42.00	23.20
408	133.20	102.39	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	102.39	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	25.15	6.12	40.24	43.62
414	133.20	85.85	24.95	6.12	40.24	43.62
415	133.20	69.16	16.83	6.12	40.24	43.62
416	133.20	69.16	16.76	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.061	0.584	0.044	0.028	0.003	0.629	#13	0.646	Not Required	Pass
2	0.003	0.226	0.155	0.060	0.033	0.339	#13	0.035	Not Required	Pass
3	0.009	0.420	0.037	0.040	0.005	0.441	#13	0.045	Not Required	Pass
4	0.008	0.422	0.104	0.042	0.026	0.500	#13	0.080	Not Required	Pass
5	0.008	0.261	0.081	0.042	0.021	0.267	#21	0.074	Not Required	Pass
6	0.015	0.683	0.134	0.070	0.036	0.757	#21	0.045	Not Required	Pass
7	0.016	0.423	0.244	0.068	0.059	0.447	#13	0.074	Not Required	Pass
8	0.006	0.121	0.224	0.041	0.024	0.234	#23	0.095	Not Required	Pass
9	0.009	0.063	0.096	0.005	0.005	0.144	#13	0.204	Not Required	Pass
10	0.017	0.644	0.235	0.064	0.051	0.711	#21	0.080	Not Required	Pass
11	0.007	0.109	0.231	0.044	0.024	0.248	#23	0.095	Not Required	Pass
12	0.002	0.459	0.265	0.105	0.052	0.698	#13	0.053	Not Required	Pass
13	0.010	0.122	0.580	0.057	0.031	0.626	#21	0.286	Not Required	Pass
14	0.007	0.107	0.569	0.054	0.031	0.586	#23	0.190	Not Required	Pass
15	0.000	0.018	0.056	0.015	0.008	0.072	#21	Not Required	Not Required	Pass
16	0.000	0.018	0.056	0.015	0.008	0.072	#21	Not Required	Not Required	Pass
101	0.078	0.745	0.005	0.035	0.000	0.780	#13	0.646	Not Required	Pass
102	0.005	0.443	0.265	0.107	0.048	0.656	#13	0.035	Not Required	Pass
103	0.015	0.677	0.093	0.067	0.014	0.746	#21	0.045	Not Required	Pass
104	0.015	0.706	0.251	0.070	0.053	0.835	#21	0.080	Not Required	Pass
105	0.015	0.420	0.260	0.067	0.065	0.468	#21	0.074	Not Required	Pass
106	0.015	0.718	0.092	0.072	0.014	0.760	#13	0.045	Not Required	Pass
107	0.015	0.446	0.245	0.071	0.062	0.481	#13	0.074	Not Required	Pass
108	0.007	0.059	0.213	0.044	0.024	0.245	#21	0.095	Not Required	Pass
109	0.021	0.055	0.059	0.002	0.000	0.116	#13	0.204	Not Required	Pass
110	0.015	0.718	0.235	0.072	0.050	0.822	#21	0.080	Not Required	Pass
111	0.007	0.074	0.222	0.044	0.024	0.242	#21	0.095	Not Required	Pass
112	0.005	0.451	0.284	0.107	0.053	0.699	#13	0.035	Not Required	Pass
113	0.011	0.179	0.602	0.058	0.032	0.749	#21	0.286	Not Required	Pass
114	0.011	0.218	0.595	0.061	0.032	0.771	#21	0.286	Not Required	Pass
115	0.013	0.347	0.315	0.045	0.025	0.633	#21	0.473	Not Required	Pass
116	0.006	0.332	0.312	0.048	0.025	0.614	#21	0.473	Not Required	Pass
201	0.078	0.756	0.000	0.035	0.000	0.790	#13	0.646	Not Required	Pass
202	0.004	0.444	0.277	0.106	0.051	0.680	#13	0.035	Not Required	Pass
203	0.015	0.709	0.091	0.071	0.013	0.759	#13	0.045	Not Required	Pass
204	0.015	0.699	0.233	0.070	0.050	0.809	#21	0.080	Not Required	Pass
205	0.015	0.440	0.244	0.070	0.061	0.472	#21	0.074	Not Required	Pass

206	0.015	0.709	0.091	0.071	0.013	0.759	#13	0.045	Not Required	Pass
207	0.015	0.440	0.244	0.070	0.061	0.472	#21	0.074	Not Required	Pass
208	0.007	0.055	0.212	0.045	0.024	0.249	#21	0.095	Not Required	Pass
209	0.019	0.047	0.060	0.001	0.000	0.109	#13	0.204	Not Required	Pass
210	0.015	0.699	0.233	0.070	0.050	0.809	#21	0.080	Not Required	Pass
211	0.007	0.064	0.218	0.046	0.024	0.255	#21	0.095	Not Required	Pass
212	0.004	0.444	0.277	0.106	0.051	0.680	#13	0.035	Not Required	Pass
213	0.011	0.217	0.563	0.058	0.031	0.757	#21	0.286	Not Required	Pass
214	0.011	0.230	0.555	0.058	0.031	0.746	#21	0.286	Not Required	Pass
215	0.013	0.265	0.315	0.046	0.024	0.551	#21	0.473	Not Required	Pass
216	0.008	0.240	0.312	0.045	0.024	0.530	#21	0.473	Not Required	Pass
301	0.078	0.745	0.005	0.035	0.000	0.780	#13	0.646	Not Required	Pass
302	0.005	0.451	0.284	0.107	0.053	0.699	#13	0.035	Not Required	Pass
303	0.015	0.718	0.092	0.072	0.014	0.760	#13	0.045	Not Required	Pass
304	0.015	0.718	0.235	0.072	0.050	0.822	#21	0.080	Not Required	Pass
305	0.015	0.446	0.245	0.071	0.062	0.481	#13	0.074	Not Required	Pass
306	0.015	0.677	0.093	0.067	0.014	0.746	#21	0.045	Not Required	Pass
307	0.015	0.420	0.260	0.067	0.065	0.468	#21	0.074	Not Required	Pass
308	0.006	0.085	0.243	0.048	0.025	0.265	#21	0.095	Not Required	Pass
309	0.021	0.055	0.059	0.002	0.000	0.116	#13	0.204	Not Required	Pass
310	0.015	0.706	0.251	0.070	0.053	0.835	#21	0.080	Not Required	Pass
311	0.007	0.106	0.249	0.045	0.025	0.254	#21	0.095	Not Required	Pass
312	0.005	0.443	0.265	0.107	0.048	0.656	#13	0.035	Not Required	Pass
313	0.011	0.179	0.603	0.058	0.032	0.749	#21	0.286	Not Required	Pass
314	0.011	0.218	0.595	0.061	0.032	0.771	#21	0.286	Not Required	Pass
315	0.013	0.268	0.316	0.044	0.024	0.554	#21	0.473	Not Required	Pass
316	0.008	0.241	0.311	0.044	0.024	0.531	#21	0.473	Not Required	Pass
401	0.061	0.584	0.044	0.028	0.003	0.629	#13	0.646	Not Required	Pass
402	0.002	0.459	0.265	0.105	0.052	0.698	#13	0.053	Not Required	Pass
403	0.015	0.683	0.134	0.070	0.036	0.757	#21	0.045	Not Required	Pass
404	0.017	0.644	0.235	0.064	0.051	0.711	#21	0.080	Not Required	Pass
405	0.016	0.423	0.244	0.068	0.059	0.447	#13	0.074	Not Required	Pass
406	0.009	0.420	0.037	0.040	0.005	0.441	#13	0.045	Not Required	Pass
407	0.008	0.261	0.081	0.042	0.021	0.267	#21	0.074	Not Required	Pass
408	0.000	0.018	0.056	0.015	0.008	0.072	#21	Not Required	Not Required	Pass
409	0.009	0.063	0.096	0.005	0.005	0.144	#13	0.204	Not Required	Pass
410	0.008	0.422	0.104	0.042	0.026	0.500	#13	0.080	Not Required	Pass
411	0.000	0.018	0.056	0.015	0.008	0.072	#21	Not Required	Not Required	Pass
412	0.003	0.226	0.155	0.060	0.033	0.339	#13	0.035	Not Required	Pass
413	0.010	0.122	0.579	0.057	0.031	0.626	#21	0.190	Not Required	Pass
414	0.007	0.107	0.570	0.054	0.031	0.586	#23	0.286	Not Required	Pass
415	0.013	0.348	0.316	0.044	0.024	0.636	#21	0.473	Not Required	Pass
416	0.006	0.347	0.309	0.041	0.024	0.624	#21	0.473	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
------------	--------------	---------

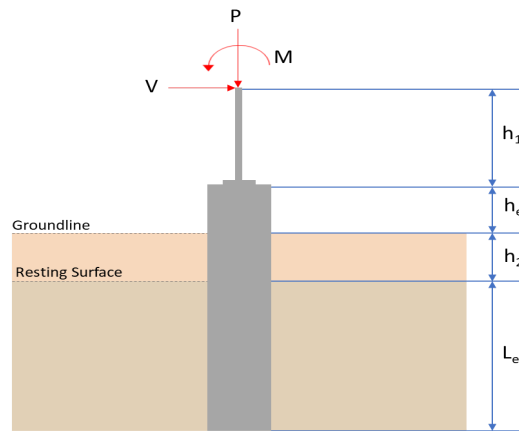
SkyCiv Foundation Design

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

$L = 7.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.990	10.569
V_x (kip)	-2.875	-4.812
V_z (kip)	0.342	0.530
M_x (kipft)	1.527	2.345
M_z (kipft)	42.896	72.391

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,

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

$$H_o = \frac{(-2.875 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4578 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.896 \text{ kipft}) + ((-2.875 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8306 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.0635 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.342 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.054459 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.527 \text{ kipft}) + ((0.342 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.24315 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.0918 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.0635 \text{ ft}), (3.0918 \text{ ft})]$$

$$L_{e,req} = 7.064 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.064 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.94187$$

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

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

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

$$q = \frac{(6.99 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.43688 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.43688 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.21844$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.4578 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8306 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (6.8306 \text{ kipft/ft})) + (4 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1569 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8306 \text{ kipft/ft})) + (3 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (6.8306 \text{ kipft/ft})) + (2 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.28354 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8306 \text{ kipft/ft})) + ((-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0909 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1569 \text{ ft})}{2}$$

$$p_a = 0.38677 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28354 \text{ kip/ft}^2)}{(0.38677 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73312$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0909 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96973$$

Status: **PASS**
Ratio: **0.730**

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

Considering z-direction:

$H_o = 0.054459 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.24315 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.24315 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.054459 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.24315 \text{ kipft/ft})) + (4 \times (0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3302 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.24315 \text{ kipft/ft})) + (3 \times (0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.24315 \text{ kipft/ft})) + (2 \times (0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.041655 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.24315 \text{ kipft/ft})) + ((0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.095439 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3302 \text{ ft})}{2}$$

$$p_a = 0.39976 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.041655 \text{ kip/ft}^2)}{(0.39976 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.1042$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

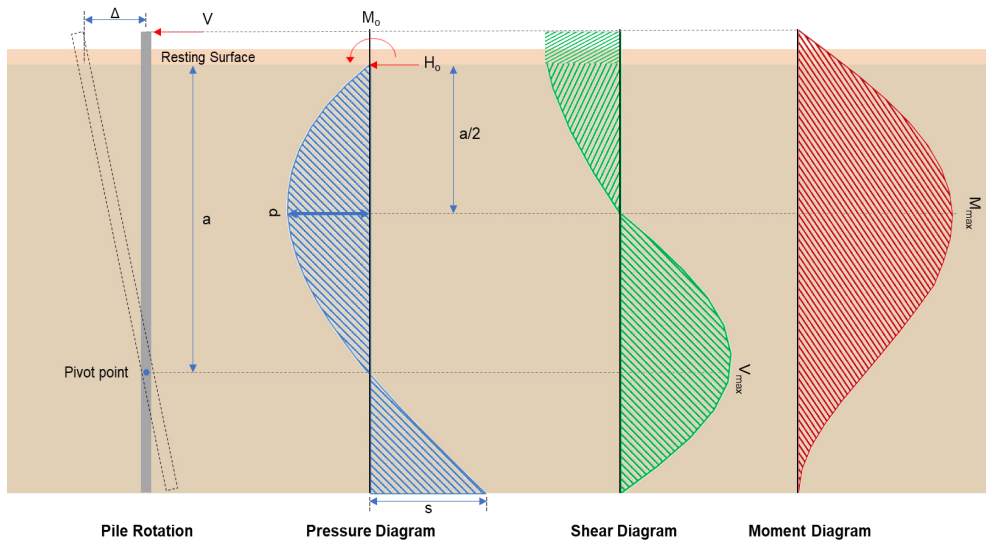
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.095439 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.084835$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.812 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.76624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(72.391 \text{ kipft}) + ((-4.812 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.527 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.527 \text{ kipft/ft})}{(-0.76624 \text{ kip/ft})}$$

$$E = 15.044 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.527 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.527 \text{ kipft/ft})) + (4 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft})}{(6 \times (11.527 \text{ kipft/ft})) + (4 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1559 \text{ ft}$$

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

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

$$V_{max} = ((-0.76624 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1559 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1559 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.902 \text{ kip}$$

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

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

$$M_{max} = ((-0.76624 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(15.044 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1559 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1559 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1559 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 46.292 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.53 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.084395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.345 \text{ kipft}) + ((0.53 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.37341 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.37341 \text{ kipft/ft})}{(0.084395 \text{ kip/ft})}$$

$$E = 4.4245 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.37341 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (0.084395 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.37341 \text{ kipft/ft})) + (4 \times (0.084395 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3316 \text{ ft}$$

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

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

$$V_{max} = ((0.084395 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.4245 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3316 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.4245 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3316 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.57677 \text{ kip}$$

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

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

$$M_{max} = ((0.084395 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(4.4245 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3316 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.4245 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3316 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.4245 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3316 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.9365 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.569 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.245 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.245 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(10.569 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0039508$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.569 \text{ kip} \rightarrow 10569 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.89 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.89 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.89 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.89 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.01 \text{ kip}$$

Considering x-direction:

V_{max} = 12.902 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.902 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.11622$$

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

Considering z-direction:

$V_{max} = 0.57677 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.57677 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.0051955$$

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

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:

$M_{max} = 46.292 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(46.292 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.18547$$

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

Considering z-direction:

$M_{max} = 1.9365 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.9365 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0077585$$

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

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

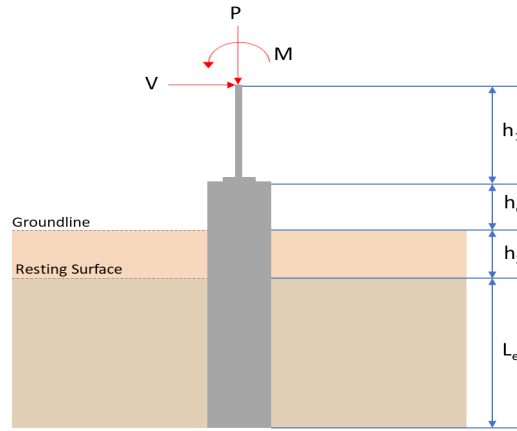
SkyCiv Foundation Design

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

$L = 7.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.990	10.569
V_x (kip)	-2.875	-4.812
V_z (kip)	-0.342	-0.530
M_x (kipft)	-1.528	-2.346
M_z (kipft)	42.896	72.392

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,

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

$$H_o = \frac{(-2.875 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4578 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.896 \text{ kipft}) + ((-2.875 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8306 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.0635 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.342 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.054459 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.528 \text{ kipft}) + ((-0.342 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.24331 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.2886 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.0635 \text{ ft}), (2.2886 \text{ ft})]$$

$$L_{e,req} = 7.064 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (7.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 7.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.064 \text{ ft})}{(7.5 \text{ ft})}$$

$$\text{Ratio} = 0.94187$$

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

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

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

$$q = \frac{(6.99 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.43688 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.43688 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.21844$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(7.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.4578 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.8306 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8306 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (6.8306 \text{ kipft/ft})) + (4 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1569 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (6.8306 \text{ kipft/ft})) + (3 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (6.8306 \text{ kipft/ft})) + (2 \times (-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = 0.28354 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (6.8306 \text{ kipft/ft})) + ((-0.4578 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 1.0909 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.1569 \text{ ft})}{2}$$

$$p_a = 0.38677 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28354 \text{ kip/ft}^2)}{(0.38677 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73312$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.0909 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96973$$

Status: **PASS**
Ratio: **0.730**

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

Considering z-direction:

$H_o = -0.054459 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.24331 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.24331 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.054459 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.24331 \text{ kipft/ft})) + (4 \times (-0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3301 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.24331 \text{ kipft/ft})) + (3 \times (-0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]^2}{(7.5 \text{ ft})^2 \times [(3 \times (0.24331 \text{ kipft/ft})) + (2 \times (-0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]}$$

$$p = -0.0097442 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.24331 \text{ kipft/ft})) + ((-0.054459 \text{ kip/ft}) \times (7.5 \text{ ft}))]}{(7.5 \text{ ft})^2}$$

$$s = 0.0083397 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.3301 \text{ ft})}{2}$$

$$p_a = 0.39975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0097442 \text{ kip/ft}^2)}{(0.39975 \text{ kip/ft}^2)}$$

$$Ratio = -0.024376$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7.5 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

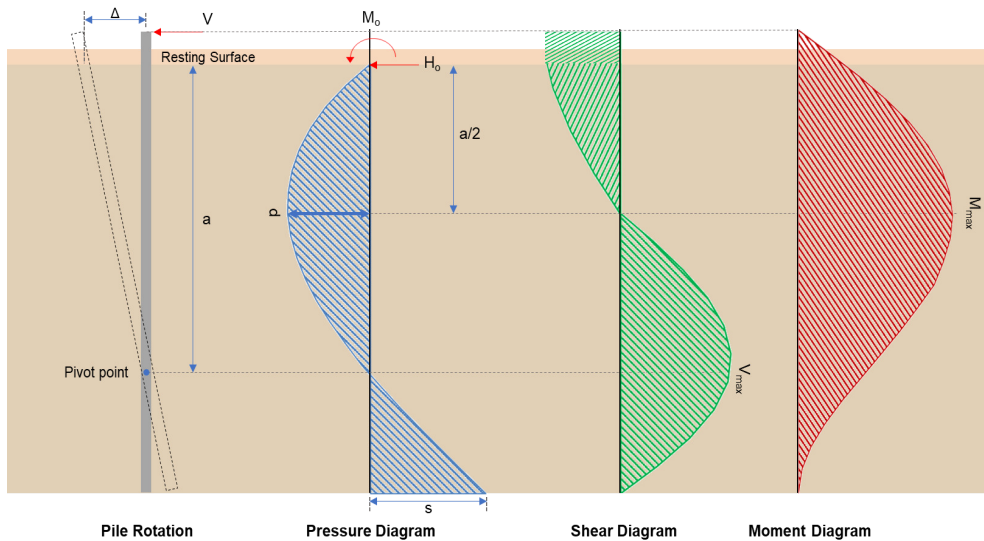
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0083397 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0074131$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.812 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.76624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(72.392 \text{ kipft}) + ((-4.812 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 11.527 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.527 \text{ kipft/ft})}{(-0.76624 \text{ kip/ft})}$$

$$E = 15.044 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.527 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (11.527 \text{ kipft/ft})) + (4 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = \frac{(-0.76624 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (11.527 \text{ kipft/ft})) + (4 \times (-0.76624 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.1559 \text{ ft}$$

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

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

$$V_{max} = ((-0.76624 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1559 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1559 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.902 \text{ kip}$$

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

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

$$M_{max} = ((-0.76624 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(15.044 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.1559 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.1559 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^3 \right] + \left[\left(\frac{3 \times (15.044 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.1559 \text{ ft})}{(2 \times (7.5 \text{ ft}))} \right)^4 \right] \right]$$

$$M_{max} = 46.293 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.53 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.084395 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.346 \text{ kipft}) + ((-0.53 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.37357 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.37357 \text{ kipft/ft})}{(-0.084395 \text{ kip/ft})}$$

$$E = 4.4264 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.37357 \text{ kipft/ft}) \times (7.5 \text{ ft})) + (3 \times (-0.084395 \text{ kip/ft}) \times (7.5 \text{ ft})^2)}{(6 \times (0.37357 \text{ kipft/ft})) + (4 \times (-0.084395 \text{ kip/ft}) \times (7.5 \text{ ft}))}$$

$$a = 5.3315 \text{ ft}$$

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

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

$$V_{max} = ((-0.084395 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.4264 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3315 \text{ ft})}{(7.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.4264 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3315 \text{ ft})}{(7.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.57691 \text{ kip}$$

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

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

$$M_{max} = ((-0.084395 \text{ kip/ft}) \times (48 \text{ in}) \times (7.5 \text{ ft})) \times \left[\left(\frac{(4.4264 \text{ ft})}{(7.5 \text{ ft})} + \frac{(5.3315 \text{ ft})}{2 \times (7.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.4264 \text{ ft})}{(7.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.3315 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.4264 \text{ ft})}{(7.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.3315 \text{ ft})}{2 \times (7.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.9371 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{10.569 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.245 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.245 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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 s_{rebar} - Minimum spacing of reinforcement,</p> <p>25.7.2.2 Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>25.7.2.1 s_{ties} - Maximum spacing of ties,</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p style="text-align: center;">$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p style="text-align: center;">$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p style="text-align: center;">$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2 ϕP_N - Allowable axial compressive strength</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</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><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.569 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0039508$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2 b_w = 48 in - Effective width, d - Effective depth</p> <p>22.5.5.1.3 λ_s - size effect modification factor</p> <p>22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</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 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 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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.569 \text{ kip} \rightarrow 10569 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10569 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.89 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.89 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.89 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.89 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.01 \text{ kip}$$

Considering x-direction:

V_{max} = 12.902 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(12.902 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.11622$$

Considering z-direction:

$V_{max} = 0.57691 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.57691 \text{ kip})}{(111.01 \text{ kip})}$$

$$Ratio = 0.0051968$$

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

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

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:

$M_{max} = 46.293 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(46.293 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.18547$$

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

Considering z-direction:

$M_{max} = 1.9371 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(1.9371 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.0077607$$

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

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

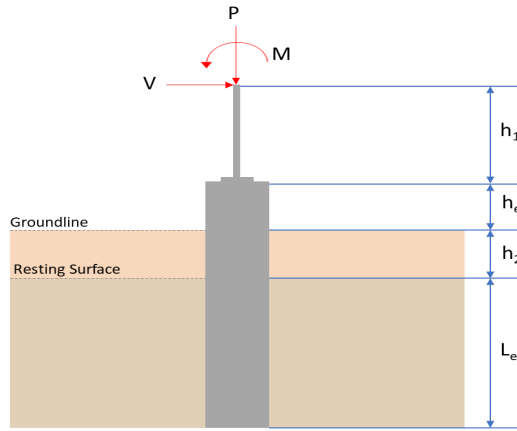
SkyCiv Foundation Design

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
 $L = 8$ ft - Total pile length
 $h_1 = 0$ ft - Lateral load height from the top of the pile,
 $h_2 = 0$ ft - Depth to resisting surface
 $h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.884	13.499
V_x (kip)	-3.617	-6.035
V_z (kip)	-0.030	-0.056
M_x (kipft)	-0.128	-0.236
M_z (kipft)	54.155	92.279

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.617 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57596 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(54.155 \text{ kipft}) + ((-3.617 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.6234 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.5432 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.128 \text{ kipft}) + ((-0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.020382 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0959 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5432 \text{ ft}), (1.0959 \text{ ft})]$$

$$L_{e,req} = 7.543 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.543 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.94288$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.884 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55525 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55525 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27763$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.57596 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.6234 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.6234 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.6234 \text{ kipft/ft})) + (4 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5084 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.6234 \text{ kipft/ft})) + (3 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.6234 \text{ kipft/ft})) + (2 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.30064 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.6234 \text{ kipft/ft})) + ((-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1849 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5084 \text{ ft})}{2}$$

$$p_a = 0.41313 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30064 \text{ kip/ft}^2)}{(0.41313 \text{ kip/ft}^2)}$$

$$Ratio = 0.72771$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1849 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.98744$$

Status: **PASS**
Ratio: **0.730**

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

Considering z-direction:

$H_o = -0.0047771 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.020382 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.020382 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.020382 \text{ kipft/ft})) + (4 \times (-0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7037 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.020382 \text{ kipft/ft})) + (3 \times (-0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 [(3 \times (0.020382 \text{ kipft/ft})) + (2 \times (-0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = -0.00084096 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.020382 \text{ kipft/ft})) + ((-0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.00023885 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7037 \text{ ft})}{2}$$

$$p_a = 0.42778 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00084096 \text{ kip/ft}^2)}{(0.42778 \text{ kip/ft}^2)}$$

$$Ratio = -0.0019659$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

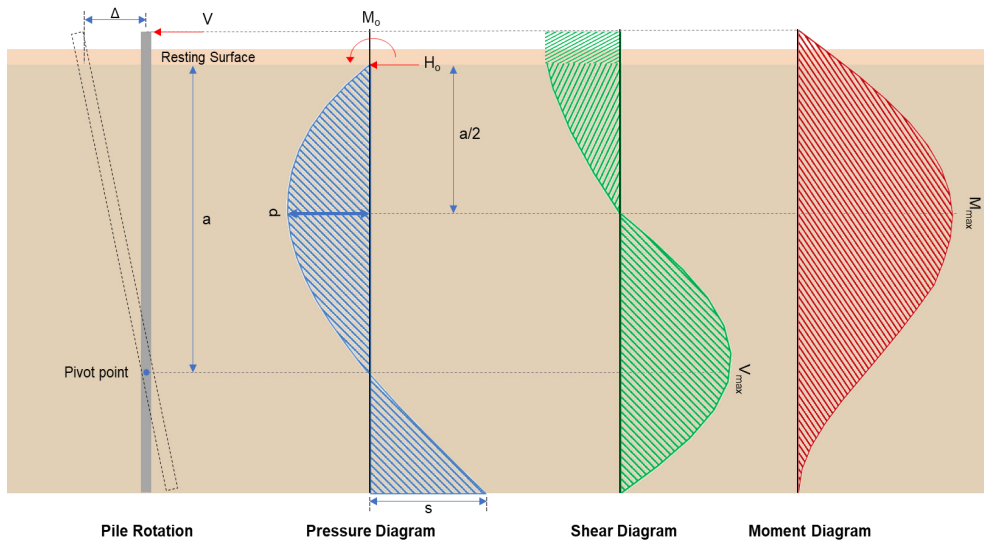
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.00023885 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.00019904$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.035 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.961 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(92.279 \text{ kipft}) + ((-6.035 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.694 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.694 \text{ kipft/ft})}{(-0.961 \text{ kip/ft})}$$

$$E = 15.291 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.694 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (14.694 \text{ kipft/ft})) + (4 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = \frac{(6 \times (14.694 \text{ kipft/ft})) + (4 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft}))}{}$$

$$a = 5.5057 \text{ ft}$$

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

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

$$V_{max} = ((-0.961 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5057 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5057 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.538 \text{ kip}$$

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

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

$$M_{max} = ((-0.961 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(15.291 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 59.354 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.056 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0089172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.236 \text{ kipft}) + ((-0.056 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.03758 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.03758 \text{ kipft/ft})}{(-0.0089172 \text{ kip/ft})}$$

$$E = 4.2143 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.03758 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.0089172 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.03758 \text{ kipft/ft})) + (4 \times (-0.0089172 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7057 \text{ ft}$$

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

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

$$V_{max} = ((-0.0089172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2143 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.2143 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.056995 \text{ kip}$$

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

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

$$M_{max} = ((-0.0089172 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2143 \text{ ft})}{(8 \text{ ft})} + \frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.2143 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2143 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.7057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.20251 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.499 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.148 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.148 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(13.499 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.005046$</p>	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.499 \text{ kip} \rightarrow 13499 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13499 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.29 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.29 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.29 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.29 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.27 \text{ kip}$$

Considering x-direction:

V_{max} = 15.538 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.538 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.13964$$

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

Considering z-direction:

$V_{max} = 0.056995 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.056995 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.00051224$$

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

$\phi M_{n,2}$

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

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

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

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:

$M_{max} = 59.354 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(59.354 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2378$$

Status: **PASS**
Ratio: **0.240**

Considering z-direction:

$M_{max} = 0.20251 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.20251 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.00081134$$

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

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

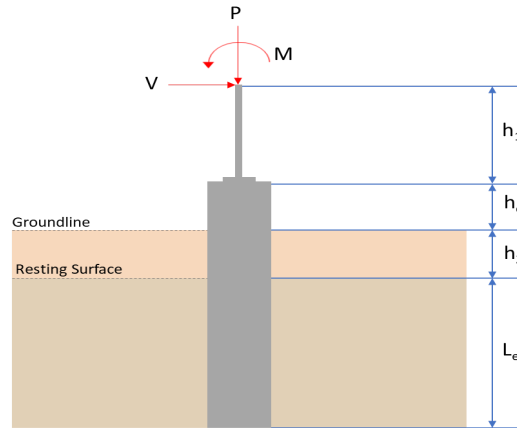
SkyCiv Foundation Design

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

$L = 8$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.884	13.499
V_x (kip)	-3.617	-6.035
V_z (kip)	0.030	0.056
M_x (kipft)	0.128	0.237
M_z (kipft)	54.155	92.279

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.617 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.57596 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(54.155 \text{ kipft}) + ((-3.617 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.6234 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 7.5432 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0047771 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.128 \text{ kipft}) + ((0.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.020382 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2581 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5432 \text{ ft}), (1.2581 \text{ ft})]$$

$$L_{e,req} = 7.543 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.543 \text{ ft})}{(8 \text{ ft})}$$

$$\text{Ratio} = 0.94288$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.884 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55525 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55525 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27763$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 2$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.57596 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.6234 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.6234 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (8.6234 \text{ kipft/ft})) + (4 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.5084 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.6234 \text{ kipft/ft})) + (3 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (8.6234 \text{ kipft/ft})) + (2 \times (-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.30064 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.6234 \text{ kipft/ft})) + ((-0.57596 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 1.1849 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.5084 \text{ ft})}{2}$$

$$p_a = 0.41313 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.30064 \text{ kip/ft}^2)}{(0.41313 \text{ kip/ft}^2)}$$

$$Ratio = 0.72771$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.1849 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.98744$$

Status: **PASS**
Ratio: **0.730**

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

Considering z-direction:

$H_o = 0.0047771 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.020382 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.020382 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.020382 \text{ kipft/ft})) + (4 \times (0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.7037 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.020382 \text{ kipft/ft})) + (3 \times (0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]^2}{(8 \text{ ft})^2 \times [(3 \times (0.020382 \text{ kipft/ft})) + (2 \times (0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]}$$

$$p = 0.0032782 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.020382 \text{ kipft/ft})) + ((0.0047771 \text{ kip/ft}) \times (8 \text{ ft}))]}{(8 \text{ ft})^2}$$

$$s = 0.0074045 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.7037 \text{ ft})}{2}$$

$$p_a = 0.42778 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0032782 \text{ kip/ft}^2)}{(0.42778 \text{ kip/ft}^2)}$$

$$Ratio = 0.0076632$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

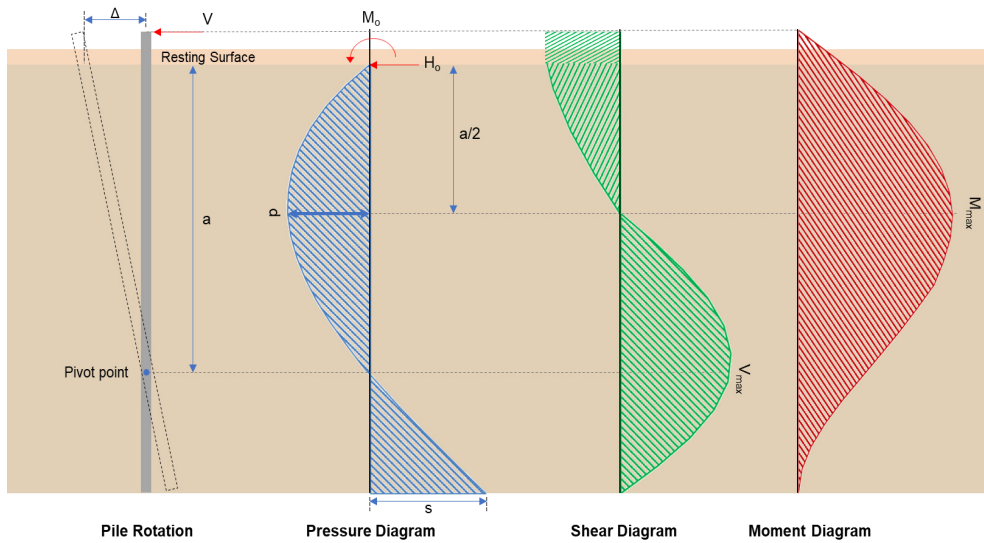
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0074045 \text{ kip/ft}^2)}{(1.2 \text{ kip/ft}^2)}$$

$$Ratio = 0.0061704$$

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,

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

$$H_o = \frac{(-6.035 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.961 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(92.279 \text{ kipft}) + ((-6.035 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.694 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.694 \text{ kipft/ft})}{(-0.961 \text{ kip/ft})}$$

$$E = 15.291 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.694 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft})^2)}{6 \times (14.694 \text{ kipft/ft}) + 4 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft})}$$

$$a = \frac{6 \times (14.694 \text{ kipft/ft}) + (4 \times (-0.961 \text{ kip/ft}) \times (8 \text{ ft}))}{\dots}$$

$$a = 5.5057 \text{ ft}$$

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

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

$$V_{max} = ((-0.961 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5057 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5057 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.538 \text{ kip}$$

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

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

$$M_{max} = ((-0.961 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(15.291 \text{ ft})}{(8 \text{ ft})} + \frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.291 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.5057 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 59.354 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.056 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0089172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.237 \text{ kipft}) + ((0.056 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.037739 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.037739 \text{ kipft/ft})}{(0.0089172 \text{ kip/ft})}$$

$$E = 4.2321 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.037739 \text{ kipft/ft}) \times (8 \text{ ft})) + (3 \times (0.0089172 \text{ kip/ft}) \times (8 \text{ ft})^2)}{(6 \times (0.037739 \text{ kipft/ft})) + (4 \times (0.0089172 \text{ kip/ft}) \times (8 \text{ ft}))}$$

$$a = 5.705 \text{ ft}$$

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

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

$$V_{max} = ((0.0089172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.2321 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.705 \text{ ft})}{(8 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.2321 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.705 \text{ ft})}{(8 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.057134 \text{ kip}$$

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

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

$$M_{max} = ((0.0089172 \text{ kip/ft}) \times (48 \text{ in}) \times (8 \text{ ft})) \times \left[\left(\frac{(4.2321 \text{ ft})}{(8 \text{ ft})} + \frac{(5.705 \text{ ft})}{2 \times (8 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.2321 \text{ ft})}{(8 \text{ ft})} + 3 \right) \times \left(\frac{(5.705 \text{ ft})}{2 \times (8 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.2321 \text{ ft})}{(8 \text{ ft})} + 2 \right) \times \left(\frac{(5.705 \text{ ft})}{2 \times (8 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.20307 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.499 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.148 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.148 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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;">$Ratio = \frac{(13.499 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.005046$</p>	<p>Status: PASS Ratio: 0.010</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.499 \text{ kip} \rightarrow 13499 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13499 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.29 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.29 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.29 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.29 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.27 \text{ kip}$$

Considering x-direction:

V_{max} = 15.538 kip - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(15.538 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.13964$$

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

Considering z-direction:

$V_{max} = 0.057134 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.057134 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.00051349$$

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:

$M_{max} = 59.354 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(59.354 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.2378$$

Status: **PASS**
Ratio: **0.240**

Considering z-direction:

$M_{max} = 0.20307 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.20307 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00081357$$

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

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

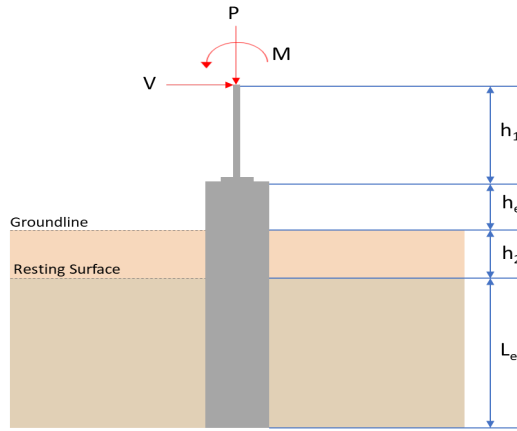
SkyCiv Foundation Design

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

$L = 8.25$ 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)	8.815	13.386
V_x (kip)	-3.645	-6.078
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	-0.001
M_z (kipft)	55.027	93.755

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.645 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.58041 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(55.027 \text{ kipft}) + ((-3.645 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 8.7623 \text{ kipft/ft}$$

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:

$L_{e,x} = 7.5872 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

$L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(7.5872 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 7.587 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(7.587 \text{ ft})}{(8.25 \text{ ft})}$$

$$\text{Ratio} = 0.91964$$

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

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

$$q = \frac{(8.815 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55094 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55094 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27547$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.58041 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 8.7623 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (8.7623 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.58041 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (8.7623 \text{ kipft/ft})) + (4 \times (-0.58041 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6836 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (8.7623 \text{ kipft/ft})) + (3 \times (-0.58041 \text{ kip/ft}) \times (8.25 \text{ ft}))]^2}{(8.25 \text{ ft})^2 \times [(3 \times (8.7623 \text{ kipft/ft})) + (2 \times (-0.58041 \text{ kip/ft}) \times (8.25 \text{ ft}))]}$$

$$p = 0.28212 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (8.7623 \text{ kipft/ft})) + ((-0.58041 \text{ kip/ft}) \times (8.25 \text{ ft}))]}{(8.25 \text{ ft})^2}$$

$$s = 1.1227 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.6836 \text{ ft})}{2}$$

$$p_a = 0.42627 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28212 \text{ kip/ft}^2)}{(0.42627 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.66184$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.25 \text{ ft})$$

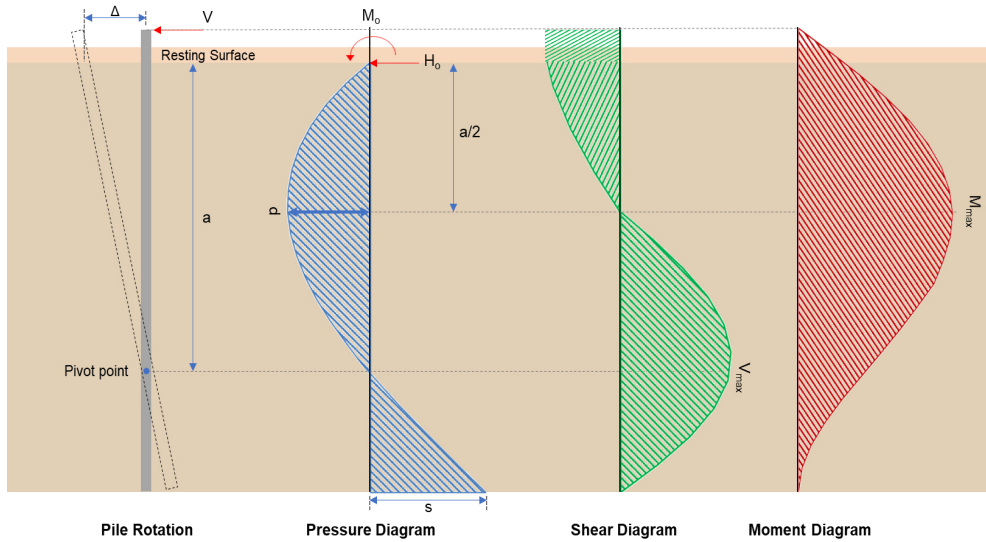
$$p_s = 1.2375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1227 \text{ kip/ft}^2)}{(1.2375 \text{ kip/ft}^2)}$$

Status: **PASS**
Ratio: **0.660**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.078 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.96783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(93.755 \text{ kipft}) + ((-6.078 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 14.929 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(14.929 \text{ kipft/ft})}{(-0.96783 \text{ kip/ft})}$$

$$E = 15.425 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (14.929 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (-0.96783 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (14.929 \text{ kipft/ft})) + (4 \times (-0.96783 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

$$a = 5.6807 \text{ ft}$$

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

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

$$V_{max} = ((-0.96783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.425 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6807 \text{ ft})}{(8.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.425 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6807 \text{ ft})}{(8.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.505 \text{ kip}$$

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

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

$$M_{max} = ((-0.96783 \text{ kip/ft}) \times (48 \text{ in}) \times (8.25 \text{ ft})) \times \left[\left(\frac{(15.425 \text{ ft})}{(8.25 \text{ ft})} + \frac{(5.6807 \text{ ft})}{2 \times (8.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.425 \text{ ft})}{(8.25 \text{ ft})} + 3 \right) \times \left(\frac{(5.6807 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (15.425 \text{ ft})}{(8.25 \text{ ft})} + 2 \right) \times \left(\frac{(5.6807 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 60.469 \text{ kipft}$$

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,

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

$$M_o = \frac{(0.001 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.00015924 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.00015924 \text{ kipft/ft}) \times (8.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (8.25 \text{ ft})^2)}{(6 \times (0.00015924 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (8.25 \text{ ft}))}$$

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

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

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

$$V_{max} = 12 \times \left(\frac{(0.00015924 \text{ kipft/ft}) \times (48 \text{ in})}{(8.25 \text{ ft})} \right) \times \left(\frac{(5.5 \text{ ft})}{(8.25 \text{ ft})} - 1 \right) \times \left(\frac{(5.5 \text{ ft})}{(8.25 \text{ ft})} \right)^2$$

$$V_{max} = 0.00013725 \text{ kip}$$

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

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

$$M_{max} = ((0.00015924 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(5.5 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^3 + \left(3 \times \frac{(5.5 \text{ ft})}{2 \times (8.25 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 0.00056617 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

Table 22.4.2.1

22.4.2.2, 10.6.1.1

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

$$A_{st,required} = Min \left[\frac{\left(\frac{13.386 \text{ kip}}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2)) \right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.151 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = Max [(-84.151 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

$$Ratio = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

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

22.4.2.2 **Axial Compression Strength (ACI 318-19, LFRD)**

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

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

$$Ratio = \frac{(13.386 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0050038$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

22.5.2.2

$$d = 0.80 D$$

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

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

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.386 \text{ kip} \rightarrow 13386 \text{ lbf}$,

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(13386 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.27 \text{ kip}$$

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

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.27 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.27 \text{ kip}$$

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)

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.27 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.26 \text{ kip}$$

Considering x-direction:

$V_{max} = 15.363 \text{ kip}$ - Maximum shear force in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.363 \text{ kip})}{(111.26 \text{ kip})}$$

$$\text{Ratio} = 0.13809$$

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

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 $\phi M_{n,2}$

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

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

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

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:

$M_{max} = 60.469 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(60.469 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.24226$$

Status: **PASS**
Ratio: **0.240**

Considering z-direction:

$M_{max} = 0.00056617 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.00056617 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 2.2683 \times 10^{-6}$$

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