

Your Project Calculations



Project Name: Butte KOA Solar Carport RevA- GS

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Butte%20KOA%20Solar%20Carport%20RevA-%20GS&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=XCbuoF5ekcZJQowTx8WJuDcHJ68E4LhrynYqEtmcJTbjikWp3l7AnnGQjP4KGgdw

Array Specification

Product:	Beam
Unique ID:	4P-19.75-8TOP-XD-72-L-5Hx11W-JI8B
Duty Classification:	XD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	11
Total Number of Modules:	55
Desired Tilt Angle:	12
Front Edge Clearance:	16
Total Array Height at Tilt:	19.58 ft
Total Frame Length:	78.75 ft
Frame Weight:	5015 lbs
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	80.85 ft
Rail Length:	208.00 in
Rail Spacing:	3.63 ft
Rail Check:	Not Checked

Support Specifications

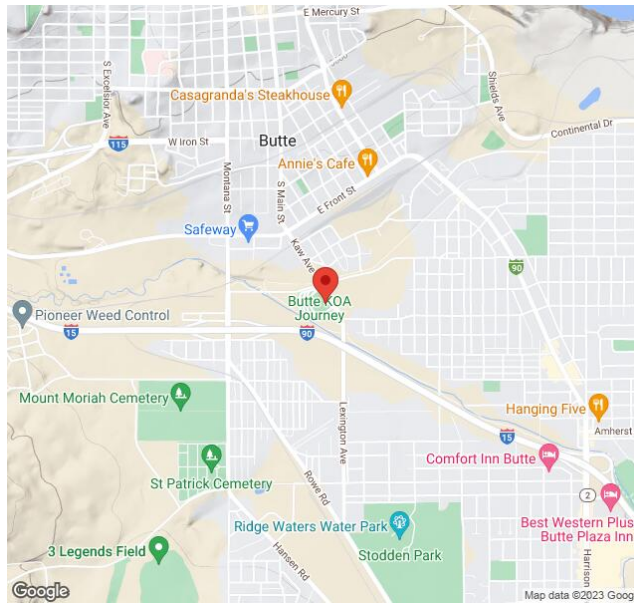
Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	17.80 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.25 ft Pile 3: 5.25 ft Pile 4: 5.25 ft
Foundation Volume:	12.444 y ³
Foundation Result:	PASSED
Mount Twist:	0.192877 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	1601 Kaw Ave, Butte, MT 59701, USA
Wind Speed:	100 mph
Snow Load:	32 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.019354 ksf



Design Disclaimer

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

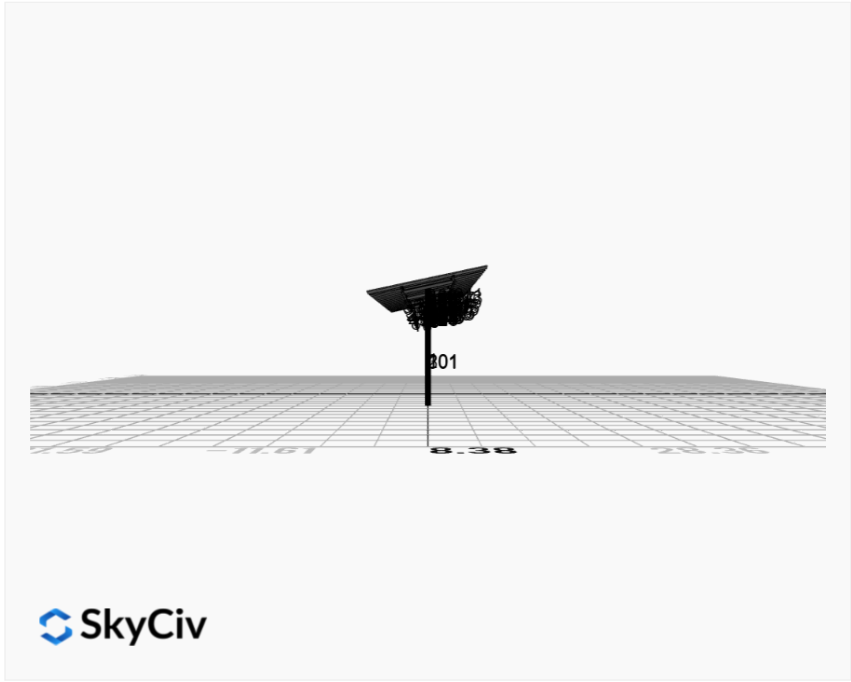
AutoDesigner Input

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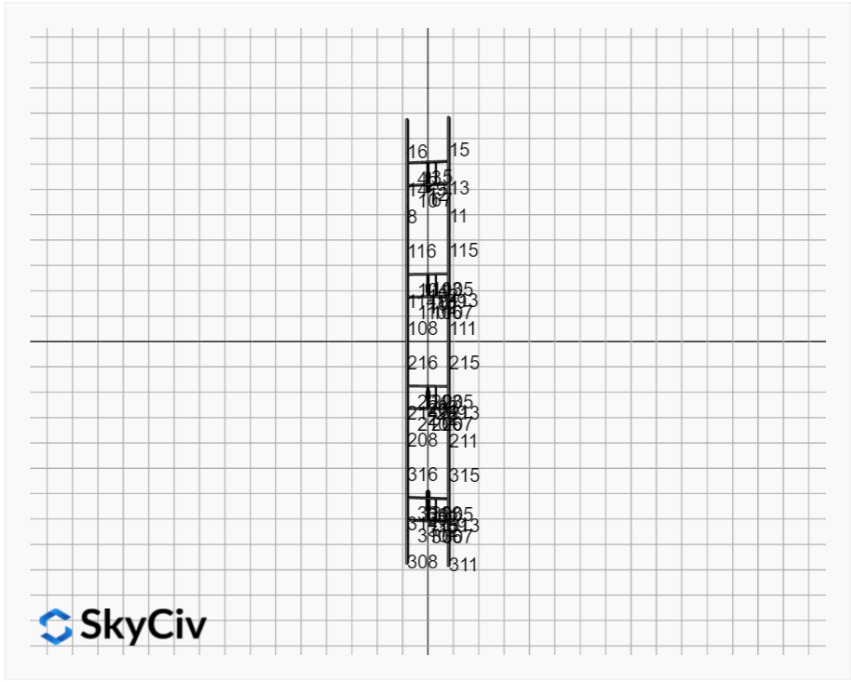
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

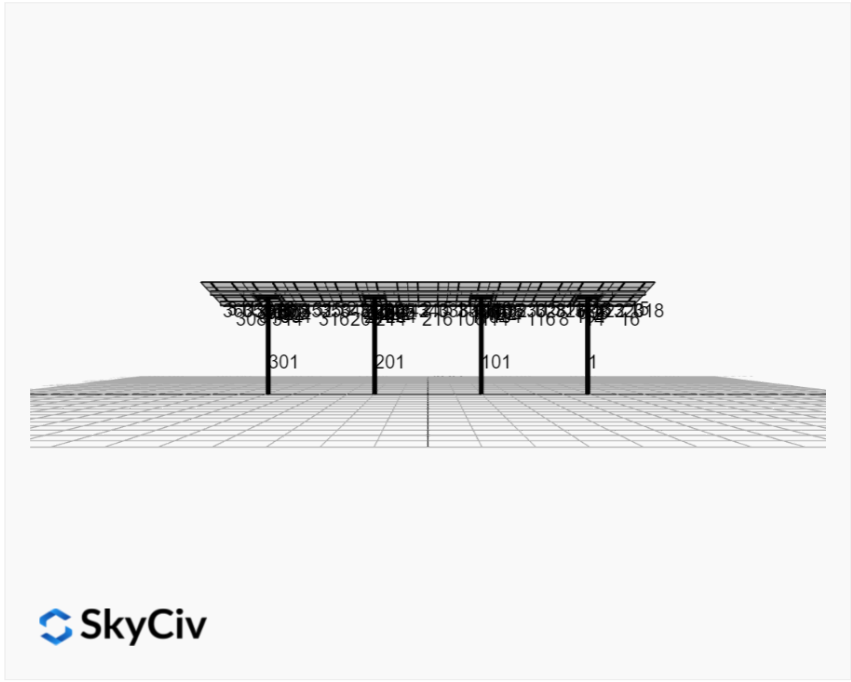
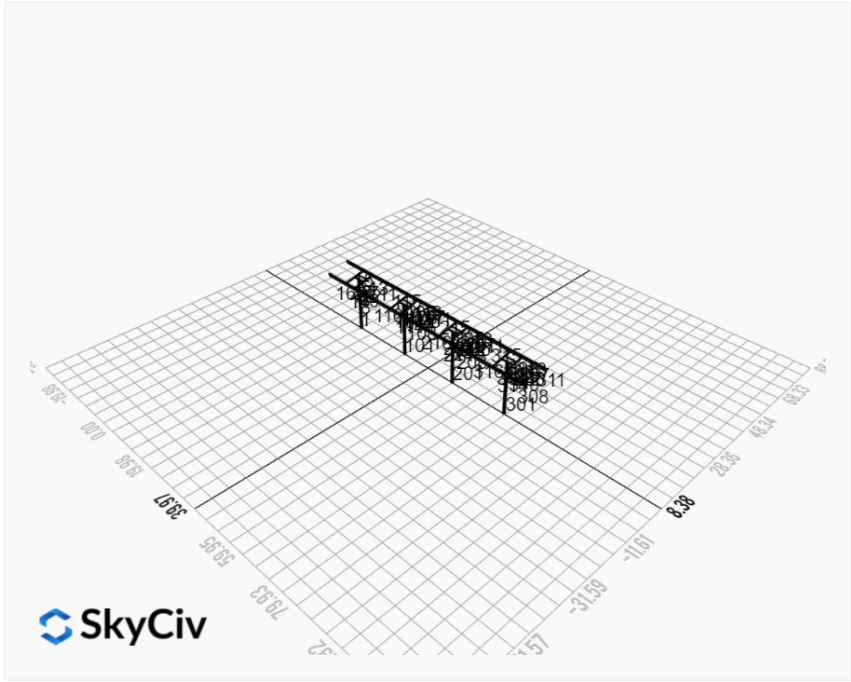




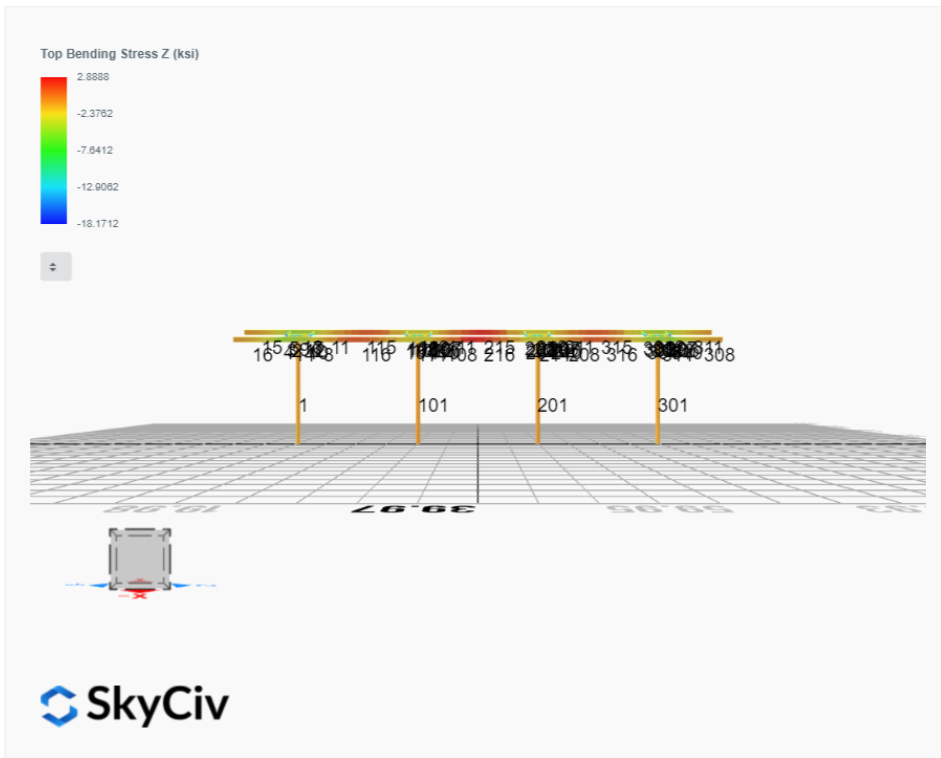
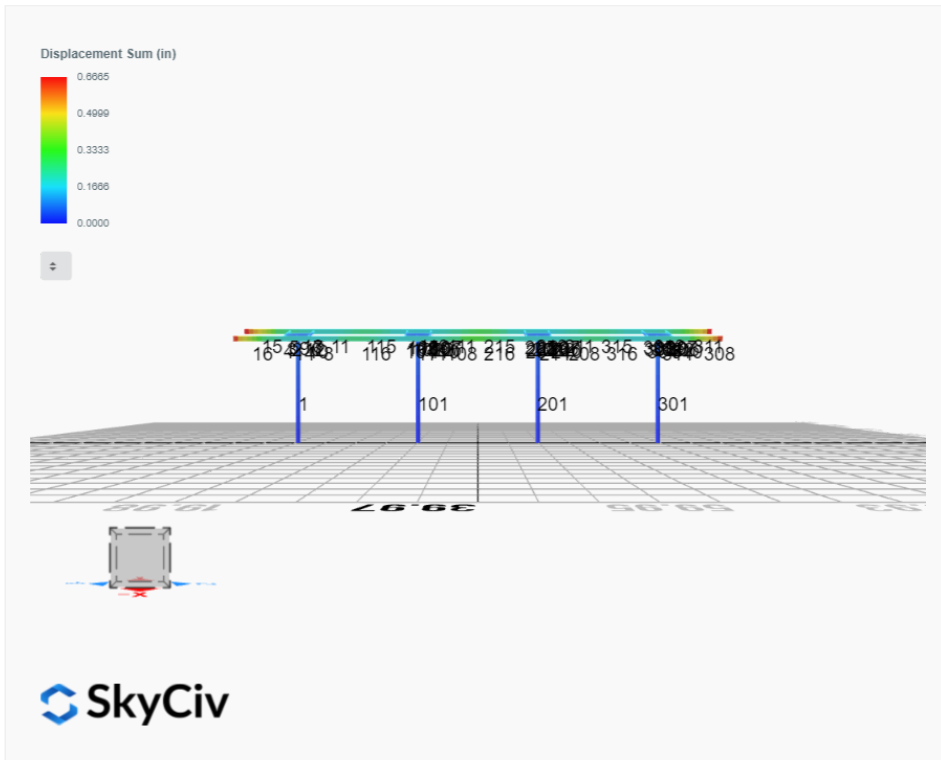
 SkyCiv

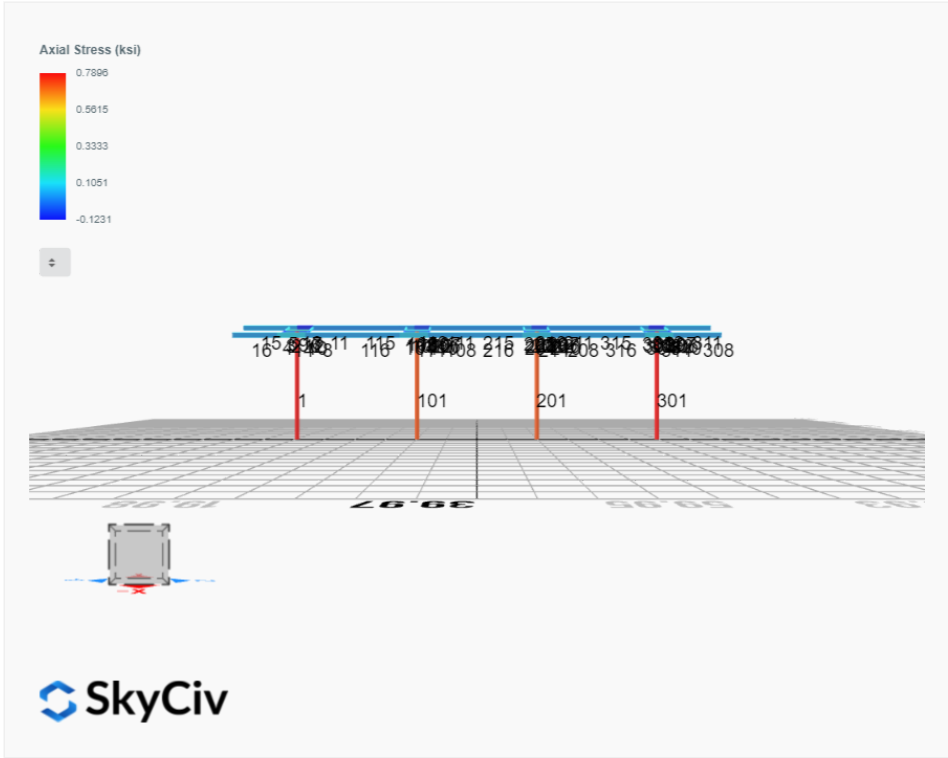


 SkyCiv



FEM Results (Envelope Worst Case for each member)





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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0086	2.8748	-0.0304	-0.1746	0.0248	0.1703
ULS: 2. D + L	-0.0086	2.8748	-0.0304	-0.1746	0.0248	0.1703
ULS: 3. D + (S or Lr or R)	-0.0353	9.5072	-0.1265	-0.7289	0.1033	0.6294
ULS: 3. D + (S or Lr or R)	-0.0086	2.8748	-0.0304	-0.1746	0.0248	0.1703
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0286	7.8491	-0.1025	-0.5904	0.0837	0.5147
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0086	2.8748	-0.0304	-0.1746	0.0248	0.1703
ULS: 5b. D + 0.7E	-0.0086	2.8748	-0.0304	-0.1746	0.0248	0.1703
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0286	7.8491	-0.1025	-0.5904	0.0837	0.5147
ULS: 8. 0.6D + 0.7E	-0.0052	1.7249	-0.0182	-0.1048	0.0149	0.1022
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5579	5.4892	-0.0698	-0.4013	0.0669	11.8705
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5579	5.4892	-0.0698	-0.4013	0.0669	11.8705
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3923	0.9760	-0.0027	-0.0163	-0.0038	-5.3698
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3343	1.2301	-0.0043	-0.0255	-0.0043	-11.8846
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4406	9.8099	-0.1320	-0.7604	0.1152	9.2899
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4406	9.8099	-0.1320	-0.7604	0.1152	9.2899
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2721	6.4250	-0.0817	-0.4716	0.0622	-3.6404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2285	6.6156	-0.0829	-0.4785	0.0618	-8.5265
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4206	4.8356	-0.0599	-0.3446	0.0564	8.9455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4206	4.8356	-0.0599	-0.3446	0.0564	8.9455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2921	1.4507	-0.0096	-0.0559	0.0033	-3.9848
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2486	1.6413	-0.0108	-0.0627	0.0030	-8.8709
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5545	4.3393	-0.0576	-0.3314	0.0569	11.8024
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5545	4.3393	-0.0576	-0.3314	0.0569	11.8024
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3958	-0.1739	0.0094	0.0536	-0.0138	-5.4379
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3377	0.0802	0.0078	0.0444	-0.0143	-11.9527

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	16.2395
Shear X	-0.9377
Shear Z	-0.2246
Moment X	-1.3034
Moment Y (Twist)	0.1927
Moment Z	21.0748

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	9.8099
Shear X	-0.5579
Shear Z	-0.1320
Moment X	-0.7604
Moment Y (Twist)	0.1152
Moment Z	11.9527

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	0.0086	2.7655	0.0061	0.0353	-0.0043	-0.1040
ULS: 2. D + L	0.0086	2.7655	0.0061	0.0353	-0.0043	-0.1040
ULS: 3. D + (S or Lr or R)	0.0353	9.0533	0.0252	0.1466	-0.0177	-0.5248
ULS: 3. D + (S or Lr or R)	0.0086	2.7655	0.0061	0.0353	-0.0043	-0.1040
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0286	7.4814	0.0205	0.1188	-0.0144	-0.4196
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0086	2.7655	0.0061	0.0353	-0.0043	-0.1040
ULS: 5b. D + 0.7E	0.0086	2.7655	0.0061	0.0353	-0.0043	-0.1040

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0286	7.4814	0.0205	0.1188	-0.0144	-0.4196
ULS: 8. 0.6D + 0.7E	0.0052	1.6593	0.0037	0.0212	-0.0026	-0.0624
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5233	5.2378	0.0139	0.0803	-0.0087	11.2353
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5233	5.2378	0.0139	0.0803	-0.0087	11.2353
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3926	0.9717	0.0000	0.0003	0.0002	-5.4380
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3465	1.2074	0.0018	0.0103	-0.0034	-11.8705
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3703	9.3356	0.0263	0.1525	-0.0177	8.0849
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3703	9.3356	0.0263	0.1525	-0.0177	8.0849
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3166	6.1360	0.0159	0.0926	-0.0110	-4.4201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2820	6.3128	0.0172	0.1001	-0.0137	-9.2445
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3903	4.6197	0.0119	0.0690	-0.0076	8.4005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3903	4.6197	0.0119	0.0690	-0.0076	8.4005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2966	1.4202	0.0015	0.0090	-0.0009	-4.1045
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2620	1.5969	0.0029	0.0166	-0.0036	-8.9289
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5267	4.1316	0.0114	0.0662	-0.0070	11.2770
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5267	4.1316	0.0114	0.0662	-0.0070	11.2770
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3891	-0.1345	-0.0024	-0.0138	0.0020	-5.3964
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3430	0.1012	-0.0007	-0.0038	-0.0017	-11.8289

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	15.4401
Shear X	-0.8865
Shear Z	0.0440
Moment X	0.2575
Moment Y (Twist)	0.0287
Moment Z	21.1607

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	9.3356
Shear X	-0.5267
Shear Z	0.0263
Moment X	0.1525
Moment Y (Twist)	0.0177
Moment Z	11.8705

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	0.0086	2.7655	-0.0061	-0.0353	0.0043	-0.1040
ULS: 2. D + L	0.0086	2.7655	-0.0061	-0.0353	0.0043	-0.1040
ULS: 3. D + (S or Lr or R)	0.0353	9.0533	-0.0252	-0.1466	0.0177	-0.5248
ULS: 3. D + (S or Lr or R)	0.0086	2.7655	-0.0061	-0.0353	0.0043	-0.1040
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0286	7.4814	-0.0205	-0.1188	0.0144	-0.4196
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0086	2.7655	-0.0061	-0.0353	0.0043	-0.1040
ULS: 5b. D + 0.7E	0.0086	2.7655	-0.0061	-0.0353	0.0043	-0.1040
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0286	7.4814	-0.0205	-0.1188	0.0144	-0.4196
ULS: 8. 0.6D + 0.7E	0.0052	1.6593	-0.0037	-0.0212	0.0026	-0.0624
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5233	5.2378	-0.0139	-0.0803	0.0087	11.2353
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5233	5.2378	-0.0139	-0.0803	0.0087	11.2353
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3926	0.9717	-0.0000	-0.0003	-0.0002	-5.4380
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3465	1.2074	-0.0018	-0.0103	0.0034	-11.8705
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3703	9.3356	-0.0263	-0.1525	0.0177	8.0849
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3703	9.3356	-0.0263	-0.1525	0.0177	8.0849
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3166	6.1360	-0.0159	-0.0926	0.0109	-4.4201
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2820	6.3128	-0.0172	-0.1001	0.0137	-9.2445

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	-0.3903	4.6197	-0.0119	-0.0690	0.0076	8.4005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3903	4.6197	-0.0119	-0.0690	0.0076	8.4005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2966	1.4202	-0.0015	-0.0090	0.0009	-4.1045
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2620	1.5969	-0.0029	-0.0166	0.0036	-8.9289
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5267	4.1316	-0.0114	-0.0662	0.0070	11.2770
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5267	4.1316	-0.0114	-0.0662	0.0070	11.2770
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3891	-0.1345	0.0024	0.0138	-0.0020	-5.3964
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3430	0.1012	0.0007	0.0038	0.0017	-11.8289

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	15.4401
Shear X	-0.8865
Shear Z	-0.0440
Moment X	-0.2574
Moment Y (Twist)	0.0286
Moment Z	21.1608

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	9.3356
Shear X	-0.5267
Shear Z	-0.0263
Moment X	-0.1525
Moment Y (Twist)	0.0177
Moment Z	11.8705

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	-0.0086	2.8748	0.0304	0.1746	-0.0248	0.1703
ULS: 2. D + L	-0.0086	2.8748	0.0304	0.1746	-0.0248	0.1703
ULS: 3. D + (S or Lr or R)	-0.0353	9.5072	0.1265	0.7290	-0.1034	0.6294
ULS: 3. D + (S or Lr or R)	-0.0086	2.8748	0.0304	0.1746	-0.0248	0.1703
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0286	7.8491	0.1025	0.5904	-0.0837	0.5146
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0086	2.8748	0.0304	0.1746	-0.0248	0.1703
ULS: 5b. D + 0.7E	-0.0086	2.8748	0.0304	0.1746	-0.0248	0.1703
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0286	7.8491	0.1025	0.5904	-0.0837	0.5146
ULS: 8. 0.6D + 0.7E	-0.0052	1.7249	0.0182	0.1048	-0.0149	0.1022
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5579	5.4892	0.0698	0.4013	-0.0669	11.8705
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5579	5.4892	0.0698	0.4013	-0.0669	11.8705
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3923	0.9760	0.0027	0.0163	0.0038	-5.3698
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3343	1.2301	0.0043	0.0255	0.0043	-11.8846
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4406	9.8099	0.1320	0.7604	-0.1153	9.2898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4406	9.8099	0.1320	0.7604	-0.1153	9.2898
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2721	6.4250	0.0817	0.4716	-0.0622	-3.6404
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2285	6.6156	0.0829	0.4785	-0.0619	-8.5265
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4206	4.8356	0.0599	0.3446	-0.0564	8.9455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4206	4.8356	0.0599	0.3446	-0.0564	8.9455
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2921	1.4507	0.0096	0.0559	-0.0033	-3.9848
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2486	1.6413	0.0108	0.0628	-0.0030	-8.8709
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5545	4.3393	0.0576	0.3314	-0.0569	11.8024
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5545	4.3393	0.0576	0.3314	-0.0569	11.8024
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3958	-0.1739	-0.0094	-0.0536	0.0138	-5.4379
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3377	0.0802	-0.0078	-0.0444	0.0143	-11.9527

Worst Case Reactions LRFD

Worst Case Reactions ASD

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	16.2395
Shear X	-0.9377
Shear Z	0.2246
Moment X	1.3036
Moment Y (Twist)	0.1929
Moment Z	21.0749

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	9.8099
Shear X	-0.5579
Shear Z	0.1320
Moment X	0.7604
Moment Y (Twist)	0.1153
Moment Z	11.9527

Project Details

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

User Name: sales@mtsolar.us
 Unit System: imperial

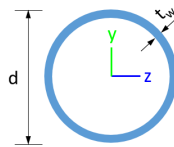


Design Input Information

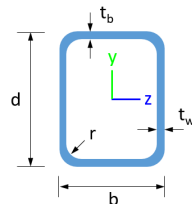
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

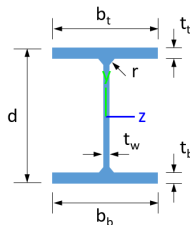
Section Dimensions



ID	Name	d (in)	t_w (in)					
3	2in Pipe Sch 120	2.38	0.25					
6	4in Pipe Sch 120	4.50	0.44					
9	8in Pipe Sch 40	8.63	0.32					



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I_{yp} (in ⁴)	I_{zp} (in ⁴)	I_w (in ⁶)	S_{yp} (in ³)	S_{zp} (in ³)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

108	20	1.33	1.33	2.0 5	1.89,1.89,1.89,1.89,1.89,1.89,1.87,1.87,1.78,2.06,1.86,1.86,2.06,2.08,1.89,1.89,1.89,1.93,1.87,1.87,1.69,2.06,1.86,1.86,2.03,2.09	3 0 0	2 0 0	1
109	3	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
110	17	2.44	2.44	3.7 5	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.69,1.68,1.67,1.67,1.63,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.75,1.69,1.67,1.67,1.65,1.69	3 0 0	2 0 0	1
111	20	1.33	1.33	2.0 5	2.06,2.06,2.06,2.06,2.06,2.06,2.06,2.06,2.06,2.09,2.06,2.06,1.32,2.08,2.06,2.06,2.06,2.02,2.06,2.06,2.10,2.06,2.06,1.78,2.08	3 0 0	2 0 0	1
112	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
113	20	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.04,1.05,1.05,1.04,1.09,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.70,1.05,1.05,1.05,1.06	3 0 0	2 0 0	1
114	20	4.88	4.00	7.5 0	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.06,1.10,1.06,1.06,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.06,1.15	3 0 0	2 0 0	1
115	20	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.24,1.17,1.17,1.13,1.18,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.17,1.32,1.17,1.17,1.16,1.18	3 0 0	2 0 0	1
116	20	6.63	6.63	10. 20	1.16,1.16,1.16,1.17,1.16,1.16,1.16,1.16,1.16,1.17,1.16,1.16,1.17,1.18,1.16,1.16,1.16,1.17,1.16,1.16,1.16,1.17,1.16,1.16,1.17,1.18	3 0 0	2 0 0	1
201	9	37.3 8	37.3 8	17. 80	-	3 0 0	2 0 0	1
202	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
203	17	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.31,1.18,1.18,1.78,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.07,1.18,1.18,1.15,1.17	3 0 0	2 0 0	1
204	17	2.44	2.44	3.7 5	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.69,1.68,1.67,1.67,1.63,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.75,1.69,1.67,1.67,1.65,1.69	3 0 0	2 0 0	1
205	17	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.91,1.67,1.67,1.24,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.56,1.67,1.67,1.63,1.66	3 0 0	2 0 0	1
206	17	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.19,1.38,1.18,1.18,2.40,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.20,1.09,1.18,1.18,1.15,1.17	3 0 0	2 0 0	1
207	17	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,2.16,1.67,1.67,1.16,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.69,1.57,1.67,1.67,1.63,1.66	3 0 0	2 0 0	1
208	20	1.33	1.33	2.0 5	2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.07,2.06,2.10,2.07,2.07,2.11,2.39,2.07,2.07,2.07,2.08,2.07,2.07,2.04,2.10,2.07,2.07,2.09,2.34	3 0 0	2 0 0	1
209	3	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
210	17	2.44	2.44	3.7 5	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.69,1.68,1.67,1.67,1.63,1.69,1.67,1.67,1.67,1.67,1.68,1.68,1.75,1.69,1.67,1.67,1.65,1.69	3 0 0	2 0 0	1
211	20	1.33	1.33	2.0 5	2.10,2.10,2.10,2.11,2.10,2.10,2.14,2.14,2.23,1.01,2.15,2.15,1.32,2.36,2.13,2.13,2.13,2.09,2.12,2.12,2.25,1.90,2.15,2.15,2.06,2.38	3 0 0	2 0 0	1
212	6	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
213	20	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.04,1.05,1.05,1.04,1.09,1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.70,1.05,1.05,1.05,1.06	3 0 0	2 0 0	1
214	20	4.88	4.00	7.5 0	1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.06,1.10,1.06,1.06,1.06,1.06,1.06,1.05,1.06,1.06,1.06,1.06,1.15	3 0 0	2 0 0	1
215	20	6.63	6.63	10. 20	1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.04,1.14,1.14,1.09,1.16,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.15,1.24,1.14,1.14,1.13,1.15	3 0 0	2 0 0	1
216	20	6.63	6.63	10. 20	1.14,1.14,1.14,1.14,1.14,1.14,1.13,1.13,1.13,1.14,1.13,1.13,1.14,1.15,1.14,1.14,1.14,1.14,1.14,1.14,1.14,1.12,1.14,1.13,1.13,1.14,1.16	3 0 0	2 0 0	1
301	9	37.3 8	37.3 8	17. 80	-	3 0 0	2 0 0	1

103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.78	6.46	56.26	44.91
114	159.30	116.35	32.09	6.46	56.26	44.91
115	159.30	75.13	21.76	6.46	56.26	44.91
116	159.30	75.13	22.34	6.46	56.26	44.91
201	377.97	81.37	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	142.47	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.78	6.46	56.26	44.91
214	159.30	116.35	32.09	6.46	56.26	44.91
215	159.30	75.13	20.03	6.46	56.26	44.91
216	159.30	75.13	21.57	6.46	56.26	44.91
301	377.97	81.37	83.29	83.29	113.39	113.39
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	21.54	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	21.54	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	116.35	33.01	6.46	56.26	44.91
314	159.30	116.35	32.70	6.46	56.26	44.91
315	159.30	75.13	25.81	6.46	56.26	44.91
316	159.30	75.13	26.00	6.46	56.26	44.91

Design Ratio

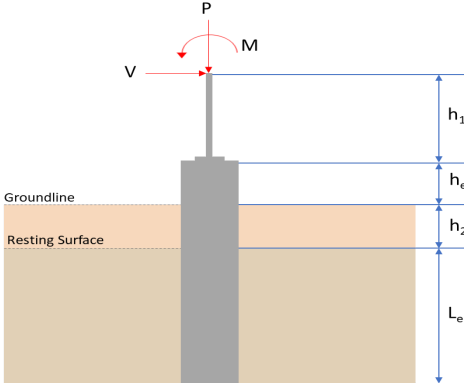
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.200	0.253	0.032	0.008	0.002	0.332	#13	0.764	Not Required	Pass
2	0.002	0.554	0.050	0.111	0.007	0.605	#21	0.036	Not Required	Pass
3	0.005	0.770	0.031	0.078	0.003	0.803	#21	0.046	Not Required	Pass
4	0.005	0.774	0.104	0.073	0.003	0.788	#21	0.046	Not Required	Pass

4	0.005	0.734	0.104	0.075	0.021	0.788	#21	0.082	Not Required	Pass
5	0.005	0.477	0.116	0.076	0.030	0.507	#21	0.076	Not Required	Pass
6	0.004	0.675	0.024	0.067	0.004	0.689	#21	0.046	Not Required	Pass
7	0.004	0.419	0.088	0.067	0.023	0.445	#21	0.076	Not Required	Pass
8	0.000	0.092	0.097	0.044	0.010	0.190	#21	0.102	Not Required	Pass
9	0.012	0.097	0.032	0.002	0.001	0.136	#21	0.206	Not Required	Pass
10	0.004	0.649	0.095	0.065	0.021	0.718	#21	0.082	Not Required	Pass
11	0.001	0.093	0.097	0.045	0.010	0.191	#21	0.068	Not Required	Pass
12	0.003	0.455	0.043	0.097	0.006	0.499	#21	0.036	Not Required	Pass
13	0.005	0.357	0.314	0.057	0.012	0.661	#21	0.204	Not Required	Pass
14	0.005	0.350	0.314	0.055	0.012	0.645	#21	0.204	Not Required	Pass
15	0.000	0.146	0.188	0.041	0.009	0.334	#21	Not Required	Not Required	Pass
16	0.000	0.140	0.188	0.039	0.009	0.328	#21	Not Required	Not Required	Pass
101	0.190	0.254	0.006	0.008	0.000	0.303	#13	0.764	Not Required	Pass
102	0.001	0.461	0.039	0.097	0.006	0.500	#21	0.036	Not Required	Pass
103	0.004	0.680	0.026	0.068	0.004	0.707	#21	0.046	Not Required	Pass
104	0.004	0.641	0.071	0.064	0.016	0.691	#21	0.082	Not Required	Pass
105	0.004	0.422	0.072	0.067	0.018	0.439	#21	0.076	Not Required	Pass
106	0.005	0.696	0.031	0.070	0.006	0.729	#21	0.046	Not Required	Pass
107	0.005	0.431	0.077	0.069	0.020	0.450	#21	0.076	Not Required	Pass
108	0.000	0.063	0.080	0.040	0.009	0.098	#21	0.102	Not Required	Pass
109	0.006	0.070	0.022	0.001	0.000	0.095	#21	0.206	Not Required	Pass
110	0.005	0.661	0.073	0.066	0.016	0.711	#21	0.082	Not Required	Pass
111	0.001	0.060	0.081	0.042	0.009	0.107	#21	0.068	Not Required	Pass
112	0.001	0.480	0.040	0.100	0.006	0.520	#21	0.036	Not Required	Pass
113	0.005	0.212	0.209	0.053	0.012	0.392	#21	0.204	Not Required	Pass
114	0.004	0.196	0.208	0.051	0.012	0.372	#21	0.306	Not Required	Pass
115	0.001	0.181	0.114	0.038	0.009	0.294	#21	0.338	Not Required	Pass
116	0.001	0.180	0.115	0.035	0.009	0.293	#21	0.507	Not Required	Pass
201	0.190	0.254	0.006	0.008	0.000	0.303	#13	0.764	Not Required	Pass
202	0.001	0.480	0.040	0.100	0.006	0.520	#21	0.036	Not Required	Pass
203	0.005	0.696	0.031	0.070	0.006	0.729	#21	0.046	Not Required	Pass
204	0.005	0.661	0.073	0.066	0.016	0.711	#21	0.082	Not Required	Pass
205	0.005	0.431	0.077	0.069	0.020	0.450	#21	0.076	Not Required	Pass
206	0.004	0.680	0.026	0.068	0.004	0.707	#21	0.046	Not Required	Pass
207	0.004	0.422	0.072	0.067	0.018	0.439	#21	0.076	Not Required	Pass
208	0.000	0.041	0.070	0.035	0.009	0.097	#24	0.102	Not Required	Pass
209	0.006	0.070	0.022	0.001	0.000	0.095	#21	0.206	Not Required	Pass
210	0.004	0.641	0.071	0.064	0.016	0.691	#21	0.082	Not Required	Pass
211	0.001	0.038	0.070	0.038	0.009	0.103	#21	0.068	Not Required	Pass
212	0.001	0.461	0.039	0.097	0.006	0.500	#21	0.036	Not Required	Pass
213	0.005	0.212	0.209	0.053	0.012	0.392	#21	0.204	Not Required	Pass
214	0.004	0.196	0.208	0.051	0.012	0.372	#21	0.306	Not Required	Pass
215	0.001	0.271	0.114	0.042	0.009	0.385	#21	0.338	Not Required	Pass
216	0.001	0.272	0.116	0.040	0.009	0.388	#21	0.507	Not Required	Pass
301	0.200	0.253	0.032	0.008	0.002	0.332	#13	0.764	Not Required	Pass
302	0.003	0.455	0.043	0.097	0.006	0.499	#21	0.036	Not Required	Pass
303	0.004	0.675	0.024	0.067	0.004	0.689	#21	0.046	Not Required	Pass
304	0.004	0.649	0.095	0.065	0.021	0.718	#21	0.082	Not Required	Pass
305	0.004	0.419	0.088	0.067	0.023	0.445	#21	0.076	Not Required	Pass
306	0.005	0.770	0.031	0.078	0.003	0.803	#21	0.046	Not Required	Pass
307	0.005	0.477	0.116	0.076	0.030	0.507	#21	0.076	Not Required	Pass
308	0.000	0.140	0.188	0.039	0.009	0.328	#21	Not Required	Not Required	Pass
309	0.012	0.097	0.032	0.002	0.001	0.136	#21	0.206	Not Required	Pass

310	0.005	0.734	0.104	0.073	0.021	0.788	#21	0.082	Not Required	Pass
311	0.000	0.146	0.188	0.041	0.009	0.334	#21	Not Required	Not Required	Pass
312	0.002	0.554	0.050	0.111	0.007	0.605	#21	0.036	Not Required	Pass
313	0.005	0.357	0.314	0.057	0.012	0.661	#21	0.204	Not Required	Pass
314	0.005	0.350	0.314	0.055	0.012	0.645	#21	0.306	Not Required	Pass
315	0.001	0.167	0.115	0.045	0.010	0.271	#21	0.338	Not Required	Pass
316	0.001	0.163	0.116	0.044	0.010	0.266	#21	0.507	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis
S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z , M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1099 1193 1189"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.810</td> <td>16.239</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.558</td> <td>-0.938</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.132</td> <td>-0.225</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.760</td> <td>-1.303</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.953</td> <td>21.075</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_n) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	9.810	16.239	V_x (kip)	-0.558	-0.938	V_z (kip)	-0.132	-0.225	M_x (kipft)	-0.760	-1.303	M_z (kipft)	11.953	21.075	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.558 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.088854 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.953 \text{ kipft}) + ((-0.558 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9033 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.0076 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.132 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.021019 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.76 \text{ kipft}) + ((-0.132 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12102 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9347 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0076 \text{ ft}), (1.9347 \text{ ft})]$$

$$L_{e,req} = 5.008 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.008 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.9539$$

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

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

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

$$q = \frac{(9.81 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.613125 \text{ kip/ft}^2$$

$$q = 0.01010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61313 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.30656$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.088854 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.9033 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9033 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.9033 \text{ kipft/ft})) + (4 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5614 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.9033 \text{ kipft/ft})) + (3 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.9033 \text{ kipft/ft})) + (2 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.21995 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.9033 \text{ kipft/ft})) + ((-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.72712 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5614 \text{ ft})}{2}$$

$$p_a = 0.26711 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21995 \text{ kip/ft}^2)}{(0.26711 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82343$$

p_s - Allowable lateral soil pressure at depth L_e ,

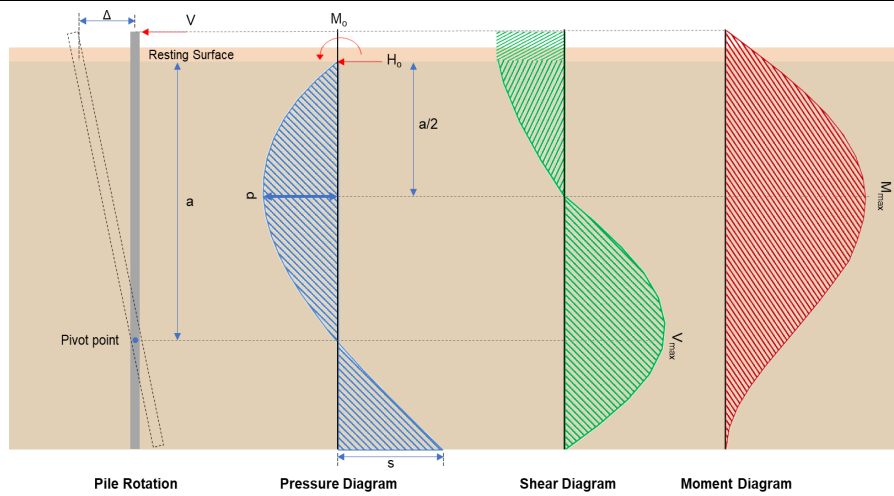
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.72712 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.92333$	Status: PASS Ratio: 0.920
	<p>Considering z-direction:</p> <p>$H_o = -0.021019 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.12102 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.12102 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.021019 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.12102 \text{ kipft/ft})) + (4 \times (-0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.6654 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.12102 \text{ kipft/ft})) + (3 \times (-0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.12102 \text{ kipft/ft})) + (2 \times (-0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = 0.004476 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.12102 \text{ kipft/ft})) + ((-0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = 0.028667 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.6654 \text{ ft})}{2}$ $p_a = 0.27491 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.004476 \text{ kip/ft}^2)}{(0.27491 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.016282$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.020

$$Ratio = \frac{(0.028667 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.036402$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.938 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14936 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.075 \text{ kipft}) + ((-0.938 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3559 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3559 \text{ kipft/ft})}{(-0.14936 \text{ kip/ft})}$$

$$E = 22.468 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3559 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.14936 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3559 \text{ kipft/ft})) + (4 \times (-0.14936 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.559 \text{ ft}$$

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

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

$$V_{max} = ((-0.14936 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.559 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.559 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9262 \text{ kip}$$

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

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

$$M_{max} = ((-0.14936 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(22.468 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.644 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.225 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.035828 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.303 \text{ kipft}) + ((-0.225 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.20748 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20748 \text{ kipft/ft})}{(-0.035828 \text{ kip/ft})}$$

$$E = 5.7911 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20748 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.035828 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.20748 \text{ kipft/ft})) + (4 \times (-0.035828 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6648 \text{ ft}$$

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

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

$$V_{max} = ((-0.035828 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.7911 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6648 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.7911 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6648 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37432 \text{ kip}$$

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

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

$$M_{max} = ((-0.035828 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.7911 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6648 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.7911 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6648 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.7911 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6648 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.9147 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(16.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.057 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.057 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.239 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0060702$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.239 \text{ kip} \rightarrow 16239 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(16239 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.65 \text{ kip}$$

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

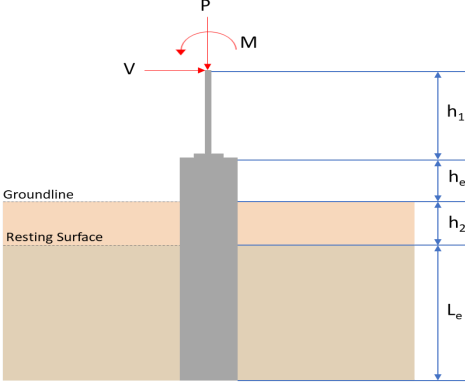
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.65 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.65 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.65 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.5 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9262 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9262 \text{ kip})}{(111.5 \text{ kip})}$ $\text{Ratio} = 0.04418$ <p>Considering z-direction:</p> <p>$V_{max} = 0.37432 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.37432 \text{ kip})}{(111.5 \text{ kip})}$ $\text{Ratio} = 0.003357$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.644 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.644 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.050656$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.9147 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.9147 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0036647$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.336</td> <td>15.440</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.527</td> <td>-0.886</td> </tr> <tr> <td>V_z (kip)</td> <td>0.026</td> <td>0.044</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.153</td> <td>0.258</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.871</td> <td>21.161</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	9.336	15.440	V_x (kip)	-0.527	-0.886	V_z (kip)	0.026	0.044	M_x (kipft)	0.153	0.258	M_z (kipft)	11.871	21.161	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.083917 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.871 \text{ kipft}) + ((-0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.8903 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.0131 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.026 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0041401 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.153 \text{ kipft}) + ((0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.024363 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3153 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0131 \text{ ft}), (1.3153 \text{ ft})]$$

$$L_{e,req} = 5.013 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.013 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.95486$$

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

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

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

$$q = \frac{(9.336 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.5835 \text{ kip/ft}^2$$

$$q = 0.0000 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.5835 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29175$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.083917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.8903 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8903 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.8903 \text{ kipft/ft})) + (4 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5588 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.8903 \text{ kipft/ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.8903 \text{ kipft/ft})) + (2 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.22117 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.8903 \text{ kipft/ft})) + ((-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.72708 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5588 \text{ ft})}{2}$$

$$p_a = 0.26691 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22117 \text{ kip/ft}^2)}{(0.26691 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82862$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.830**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.72708 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92327$$

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

Considering z-direction:

$H_o = 0.0041401 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.024363 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.024363 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.024363 \text{ kipft/ft})) + (4 \times (0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6632 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.024363 \text{ kipft/ft})) + (3 \times (0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.024363 \text{ kipft/ft})) + (2 \times (0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.0061766 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.024363 \text{ kipft/ft})) + ((0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.015339 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6632 \text{ ft})}{2}$$

$$p_a = 0.27474 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0061766 \text{ kip/ft}^2)}{(0.27474 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.022482$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

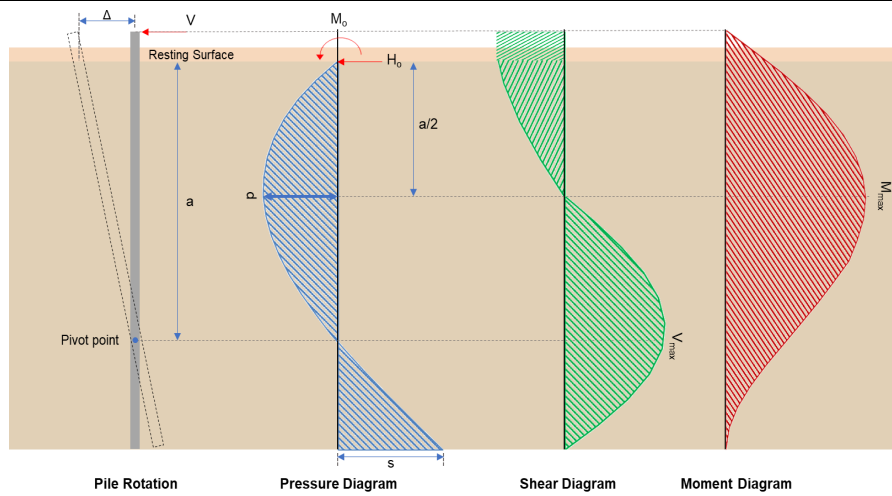
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.015339 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.019478$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.886 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.161 \text{ kipft}) + ((-0.886 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3696 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3696 \text{ kipft/ft})}{(-0.14108 \text{ kip/ft})}$$

$$E = 23.884 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3696 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.14108 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3696 \text{ kipft/ft})) + (4 \times (-0.14108 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5559 \text{ ft}$$

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

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

$$V_{max} = ((-0.14108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5559 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5559 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9234 \text{ kip}$$

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

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

$$M_{max} = ((-0.14108 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(23.884 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.652 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.258 \text{ kipft}) + ((0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.041083 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.041083 \text{ kipft/ft})}{(0.0070064 \text{ kip/ft})}$$

$$E = 5.8636 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.041083 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0070064 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.041083 \text{ kipft/ft})) + (4 \times (0.0070064 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6635 \text{ ft}$$

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

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

$$V_{max} = ((0.0070064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.8636 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6635 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.8636 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6635 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.073883 \text{ kip}$$

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

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

$$M_{max} = ((0.0070064 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.8636 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.8636 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.8636 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6635 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.18067 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.44 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.083 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.083 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.44 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0057716$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.44 \text{ kip} \rightarrow 15440 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(15440 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

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

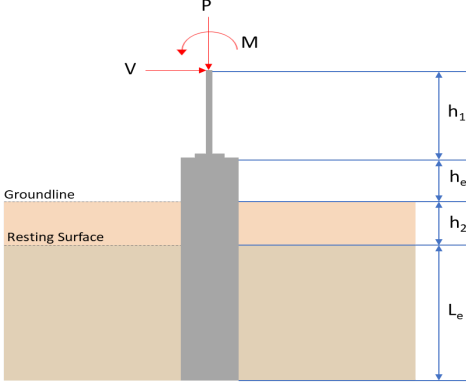
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.43 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9234 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9234 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.044182$ <p>Considering z-direction:</p> <p>$V_{max} = 0.073883 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.073883 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.00066302$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.652 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.652 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.05069$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.18067 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.18067 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00072382$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.336</td> <td>15.440</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.527</td> <td>-0.886</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.026</td> <td>-0.044</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.153</td> <td>-0.257</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.871</td> <td>21.161</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	9.336	15.440	V_x (kip)	-0.527	-0.886	V_z (kip)	-0.026	-0.044	M_x (kipft)	-0.153	-0.257	M_z (kipft)	11.871	21.161	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.527 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.083917 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.871 \text{ kipft}) + ((-0.527 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.8903 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.0131 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.026 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0041401 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.153 \text{ kipft}) + ((-0.026 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.024363 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1829 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0131 \text{ ft}), (1.1829 \text{ ft})]$$

$$L_{e,req} = 5.013 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.013 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.95486$$

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

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

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

$$q = \frac{(9.336 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.5835 \text{ kip/ft}^2$$

$$q = 0.0000 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.5835 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$Ratio = 0.29175$$

Status: **PASS**
Ratio: **0.290**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.083917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.8903 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8903 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.8903 \text{ kipft/ft})) + (4 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5588 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.8903 \text{ kipft/ft})) + (3 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.8903 \text{ kipft/ft})) + (2 \times (-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.22117 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.8903 \text{ kipft/ft})) + ((-0.083917 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.72708 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5588 \text{ ft})}{2}$$

$$p_a = 0.26691 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22117 \text{ kip/ft}^2)}{(0.26691 \text{ kip/ft}^2)}$$

$$Ratio = 0.82862$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.830**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.72708 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92327$$

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

Considering z-direction:

$H_o = -0.0041401 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.024363 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.024363 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.024363 \text{ kipft/ft})) + (4 \times (-0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6632 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.024363 \text{ kipft/ft})) + (3 \times (-0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.024363 \text{ kipft/ft})) + (2 \times (-0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.00095526 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.024363 \text{ kipft/ft})) + ((-0.0041401 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.0058755 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6632 \text{ ft})}{2}$$

$$p_a = 0.27474 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00095526 \text{ kip/ft}^2)}{(0.27474 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.003477$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

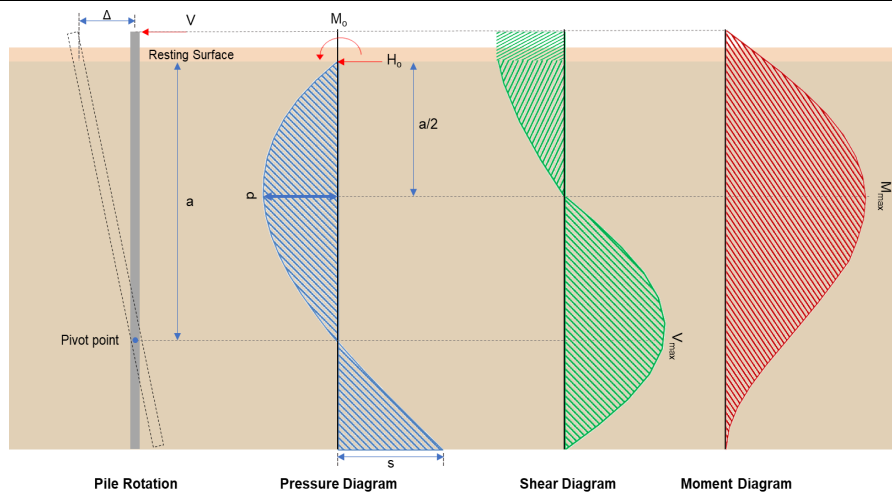
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.0058755 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0074609$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.886 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.161 \text{ kipft}) + ((-0.886 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3696 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3696 \text{ kipft/ft})}{(-0.14108 \text{ kip/ft})}$$

$$E = 23.884 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3696 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.14108 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3696 \text{ kipft/ft})) + (4 \times (-0.14108 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5559 \text{ ft}$$

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

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

$$V_{max} = ((-0.14108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5559 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5559 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9234 \text{ kip}$$

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

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

$$M_{max} = ((-0.14108 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(23.884 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.884 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.652 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.257 \text{ kipft}) + ((-0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.040924 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.040924 \text{ kipft/ft})}{(-0.0070064 \text{ kip/ft})}$$

$$E = 5.8409 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.040924 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0070064 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.040924 \text{ kipft/ft})) + (4 \times (-0.0070064 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6639 \text{ ft}$$

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

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

$$V_{max} = ((-0.0070064 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.8409 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6639 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.8409 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6639 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.073669 \text{ kip}$$

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

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

$$M_{max} = ((-0.0070064 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.8409 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6639 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.8409 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6639 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.8409 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6639 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.1801 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.44 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.083 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.083 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.44 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0057716$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 15.44 \text{ kip} \rightarrow 15440 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(15440 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.54 \text{ kip}$$

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

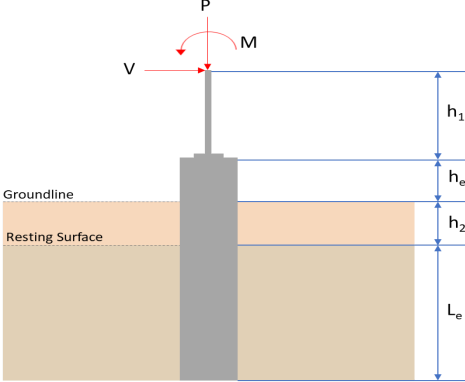
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.54 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.54 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.43 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9234 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9234 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.044182$ <p>Considering z-direction:</p> <p>$V_{max} = 0.073669 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.073669 \text{ kip})}{(111.43 \text{ kip})}$ $\text{Ratio} = 0.0006611$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.652 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.652 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.05069$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.1801 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.1801 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00072157$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.810</td> <td>16.239</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.558</td> <td>-0.938</td> </tr> <tr> <td>V_z (kip)</td> <td>0.132</td> <td>0.225</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.760</td> <td>1.304</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.953</td> <td>21.075</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	9.810	16.239	V_x (kip)	-0.558	-0.938	V_z (kip)	0.132	0.225	M_x (kipft)	0.760	1.304	M_z (kipft)	11.953	21.075	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.558 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.088854 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.953 \text{ kipft}) + ((-0.558 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.9033 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.0076 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.132 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.021019 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.76 \text{ kipft}) + ((0.132 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12102 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.3281 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0076 \text{ ft}), (2.3281 \text{ ft})]$$

$$L_{e,req} = 5.008 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.008 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.9539$$

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

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

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

$$q = \frac{(9.81 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.613125 \text{ kip/ft}^2$$

$$q = 0.01010 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61313 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.30656$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.088854 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.9033 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.9033 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.9033 \text{ kipft/ft})) + (4 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5614 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.9033 \text{ kipft/ft})) + (3 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.9033 \text{ kipft/ft})) + (2 \times (-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.21995 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.9033 \text{ kipft/ft})) + ((-0.088854 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.72712 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5614 \text{ ft})}{2}$$

$$p_a = 0.26711 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21995 \text{ kip/ft}^2)}{(0.26711 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82343$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.820**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.72712 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.92333$$

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

Considering z-direction:

$H_o = 0.021019 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.12102 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12102 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.021019 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.12102 \text{ kipft/ft})) + (4 \times (0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6654 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.12102 \text{ kipft/ft})) + (3 \times (0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.12102 \text{ kipft/ft})) + (2 \times (0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.030971 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.12102 \text{ kipft/ft})) + ((0.021019 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.07671 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6654 \text{ ft})}{2}$$

$$p_a = 0.27491 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.030971 \text{ kip/ft}^2)}{(0.27491 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.11266$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

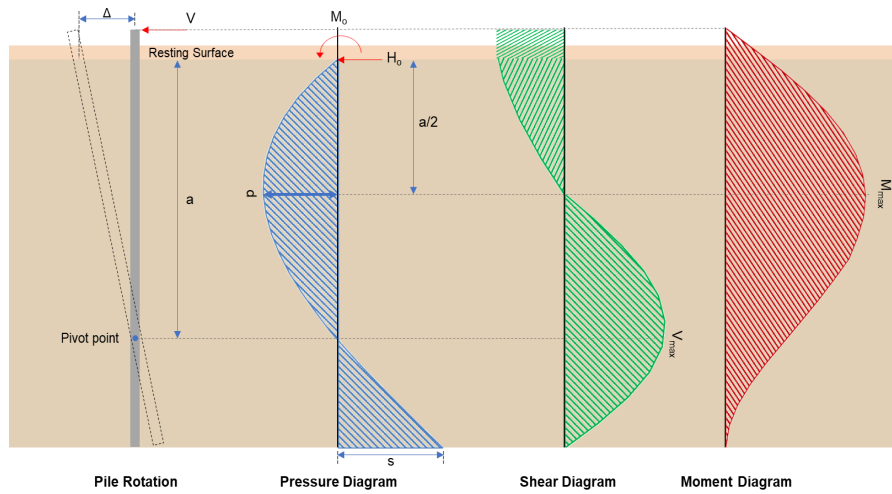
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.07671 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.09741$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.938 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14936 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(21.075 \text{ kipft}) + ((-0.938 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3559 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3559 \text{ kipft/ft})}{(-0.14936 \text{ kip/ft})}$$

$$E = 22.468 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3559 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.14936 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3559 \text{ kipft/ft})) + (4 \times (-0.14936 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.559 \text{ ft}$$

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

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

$$V_{max} = ((-0.14936 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.559 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.559 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9262 \text{ kip}$$

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

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

$$M_{max} = ((-0.14936 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(22.468 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (22.468 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.559 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.644 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.225 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.035828 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.304 \text{ kipft}) + ((0.225 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.20764 \text{ kipft/ft})}{(0.035828 \text{ kip/ft})}$$

$$E = 5.7956 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.20764 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.035828 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.20764 \text{ kipft/ft})) + (4 \times (0.035828 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6647 \text{ ft}$$

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

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

$$V_{max} = ((0.035828 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.7956 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6647 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.7956 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6647 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.37453 \text{ kip}$$

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

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

$$M_{max} = ((0.035828 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.7956 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.7956 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.7956 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6647 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91526 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(16.239 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.057 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.057 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(16.239 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0060702$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 16.239 \text{ kip} \rightarrow 16239 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(16239 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.65 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.65 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.65 \text{ kip}$$

<p>22.5.1.2</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>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.65 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.5 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9262 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9262 \text{ kip})}{(111.5 \text{ kip})}$ $\text{Ratio} = 0.04418$ <p>Considering z-direction:</p> <p>$V_{max} = 0.37453 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.37453 \text{ kip})}{(111.5 \text{ kip})}$ $\text{Ratio} = 0.0033589$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.644 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.644 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.050656$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.91526 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.91526 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0036669$	<p>Status: PASS Ratio: 0.000</p>