

Your Project Calculations



Project Name: D50Schools-Manor-JB-Area1

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=D50Schools-Manor-JB-Area1&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=AnjOEb1sbZvtYtjNXtWpuAAWsGRZC4rpADeYJEcJMIclRFqPdHofE3GD65WKmd4P

Array Specification

Product:	Beam
Unique ID:	5P-22.5-6TOP-HD-45-L-5Hx14W-L8DK
Duty Classification:	HD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	5
Number of Columns:	14
Total Number of Modules:	70
Desired Tilt Angle:	10
Front Edge Clearance:	14
Total Array Height at Tilt:	17.25 ft
Total Frame Length:	105.00 ft
Frame Weight:	4659 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	105.80 ft
Rail Length:	225.75 in
Rail Spacing:	3.78 ft
Rail Check:	PASS (69% utilized)

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	15.63 ft
Number of Poles:	5
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.25 ft Pile 2: 5.25 ft Pile 3: 5.25 ft Pile 4: 5.25 ft Pile 5: 5.25 ft
Foundation Volume:	15.556 y ³
Foundation Result:	PASSED
Mount Twist:	0.139573 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	Washington, IL 61571, USA
Wind Speed:	100 mph
Snow Load:	20 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.012096 ksf



Design Disclaimer

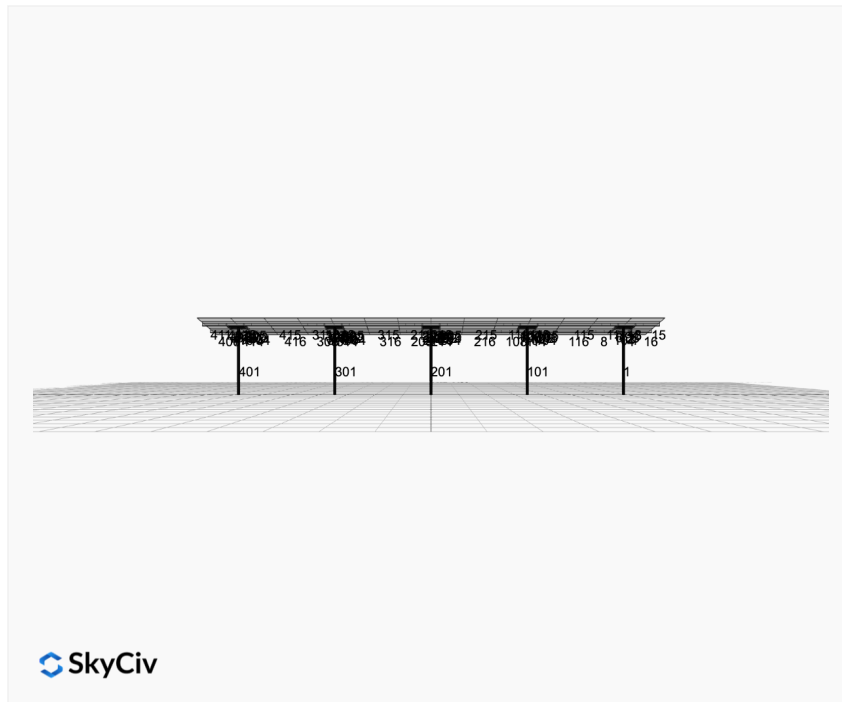
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

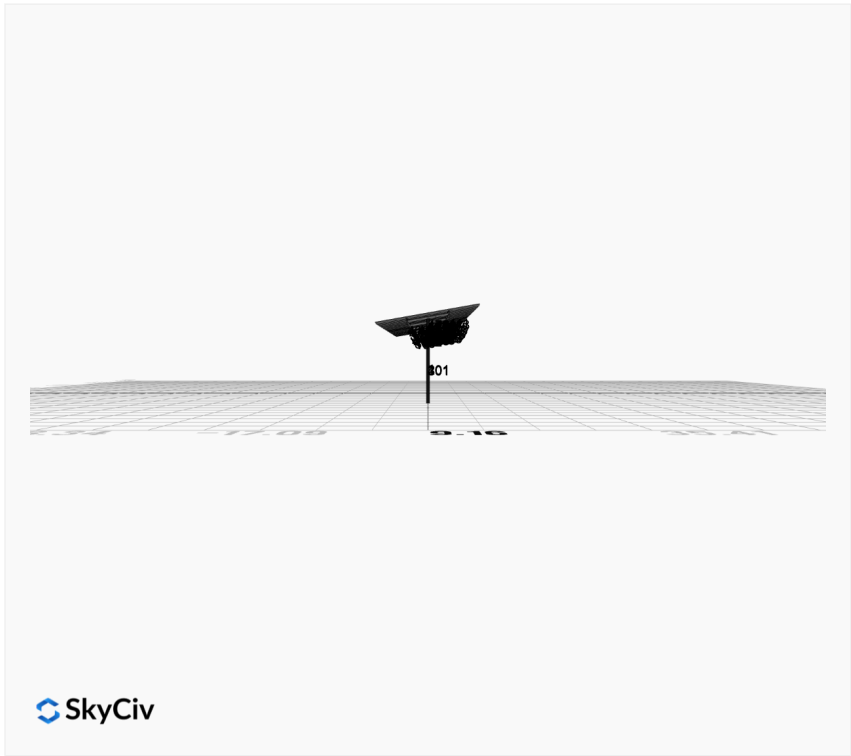
AutoDesigner Input

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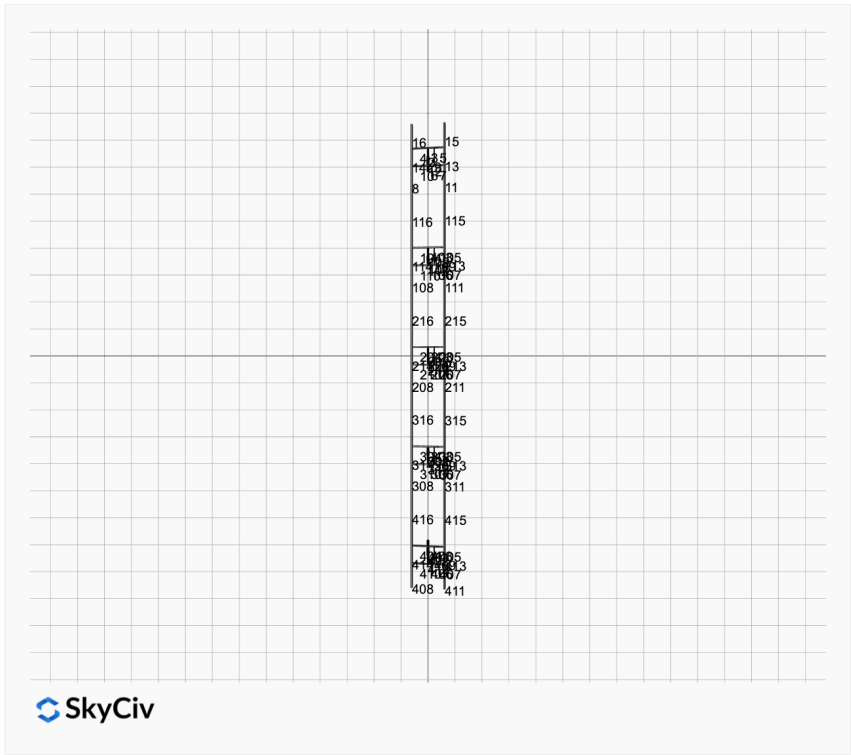
Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles

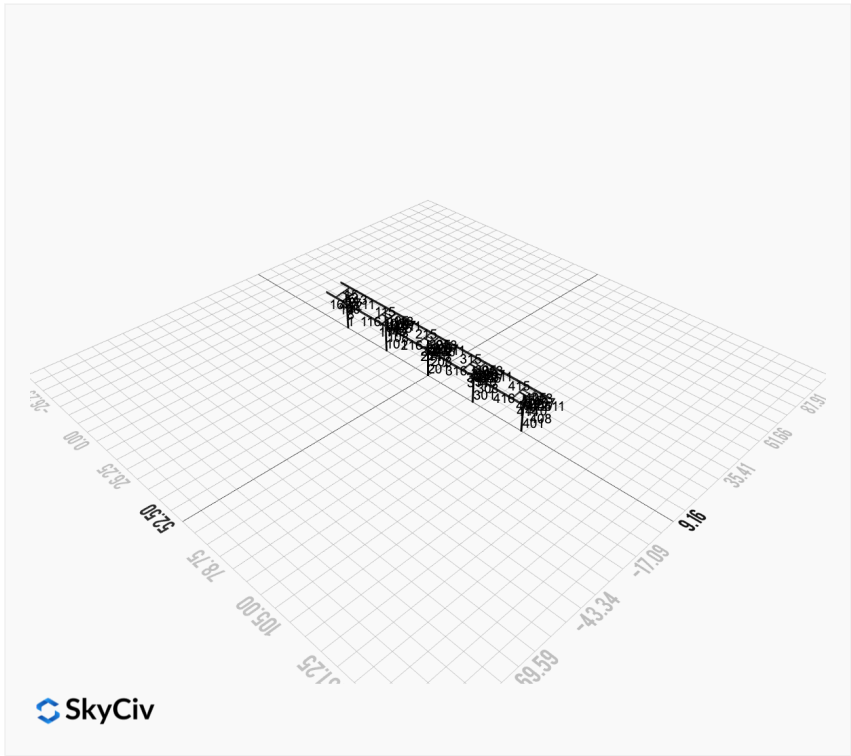




SkyCiv



SkyCiv



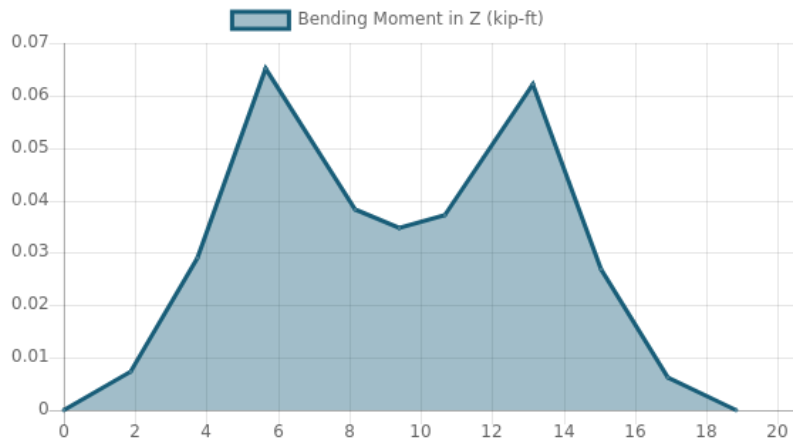
Rail Design Check

Rail Length: 18.8125 ft
Additional Restraints Required: None
Tributary Width: 3.77875 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0450 kip/ft
Snow (Y): -0.0079 kip/ft
Wind uplift Case A: 0.0435 kip/ft
Wind uplift Case A: 0.0435 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0620 kip/ft

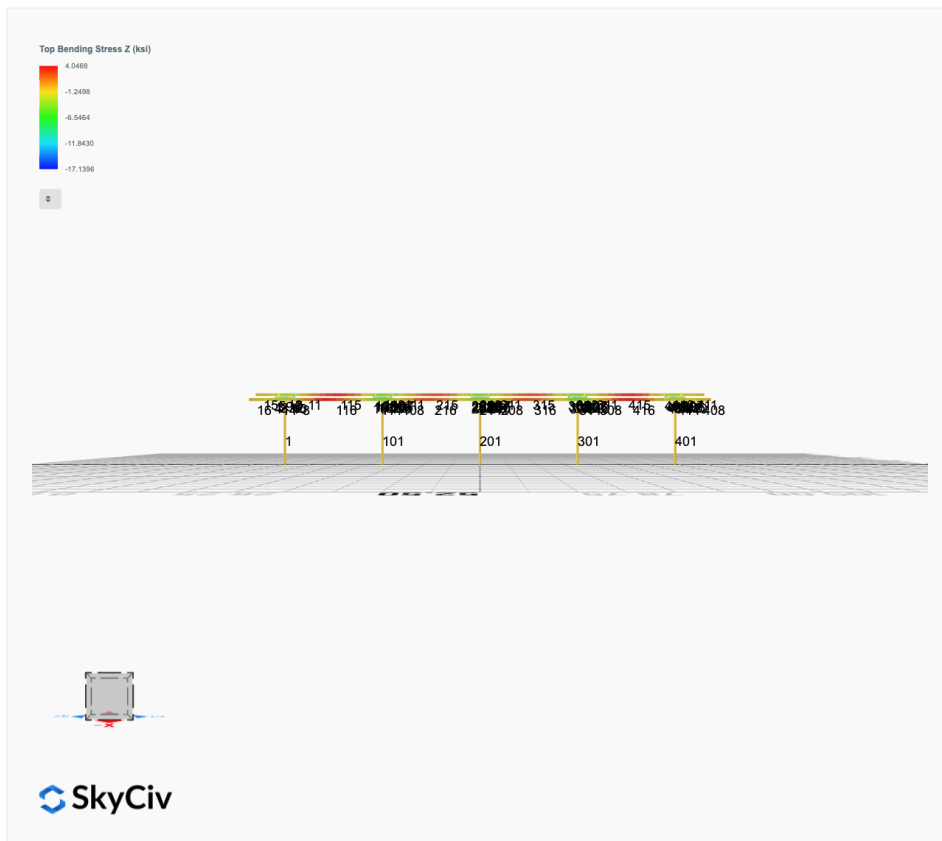
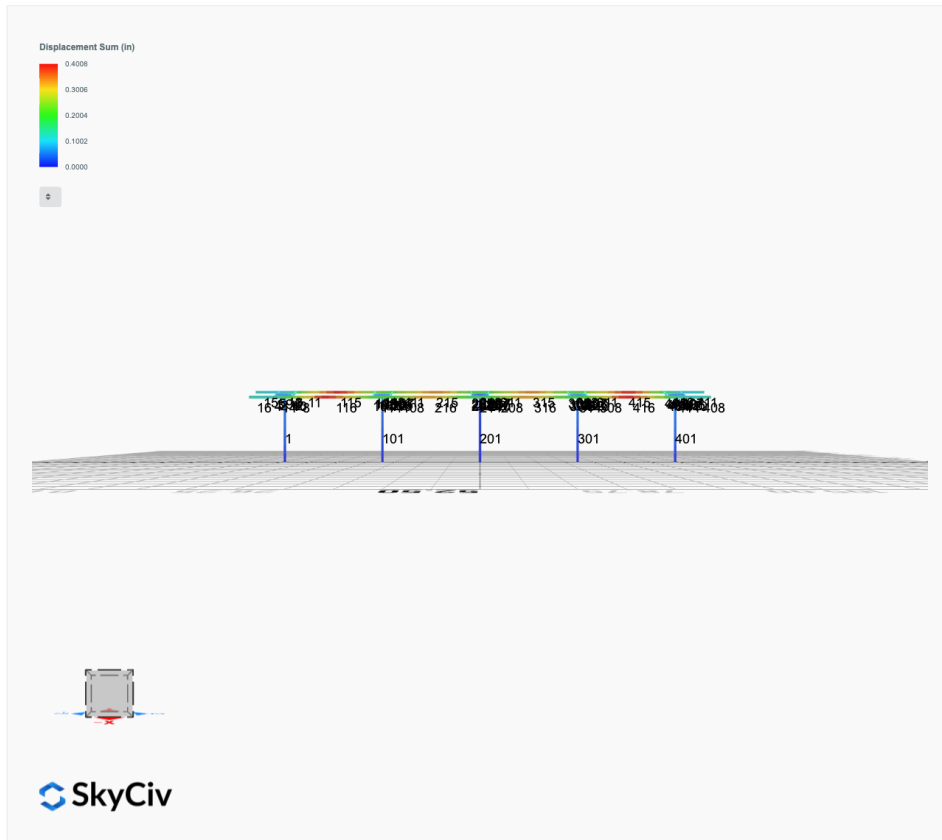


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	23.72751774	0.688	PASS
Material Yield	34.5	23.72751774	0.688	PASS
Material Strength	37	23.72751774	0.641	PASS

Member 1, ULS: 1. 1.4D



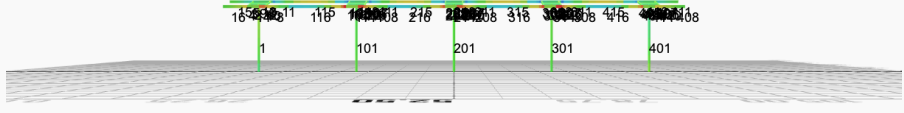
FEM Results (Envelope Worst Case for each member)



Top Bending Stress Y (ksi)



5

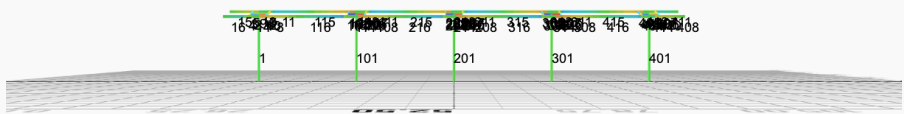


SkyCiv

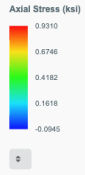
Shear Stress Y (ksi)



5



SkyCiv



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0071	2.3974	0.0470	0.2406	-0.0144	-0.0685
ULS: 2. D + L	0.0071	2.3974	0.0470	0.2406	-0.0144	-0.0685
ULS: 3. D + (S or Lr or R)	0.0222	6.4680	0.1488	0.7632	-0.0457	-0.2752
ULS: 3. D + (S or Lr or R)	0.0071	2.3974	0.0470	0.2406	-0.0144	-0.0685
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0184	5.4503	0.1233	0.6326	-0.0378	-0.2235
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.3974	0.0470	0.2406	-0.0144	-0.0685
ULS: 5b. D + 0.7E	0.0071	2.3974	0.0470	0.2406	-0.0144	-0.0685
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0184	5.4503	0.1233	0.6326	-0.0378	-0.2235
ULS: 8. 0.6D + 0.7E	0.0043	1.4384	0.0282	0.1443	-0.0086	-0.0411
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5109	5.1349	0.1268	0.6486	-0.0743	11.0341
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5109	5.1349	0.1268	0.6486	-0.0743	11.0341
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3539	0.4814	-0.0030	-0.0126	0.0180	-3.7374
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3372	0.7237	-0.0063	-0.0288	0.0300	-11.2553
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3701	7.5034	0.1832	0.9386	-0.0828	8.1033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3701	7.5034	0.1832	0.9386	-0.0828	8.1033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2785	4.0133	0.0858	0.4427	-0.0135	-2.9753
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2660	4.1950	0.0834	0.4305	-0.0045	-8.6137
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3814	4.4505	0.1069	0.5466	-0.0593	8.2584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3814	4.4505	0.1069	0.5466	-0.0593	8.2584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2672	0.9604	0.0095	0.0507	0.0099	-2.8202
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2547	1.1421	0.0071	0.0385	0.0189	-8.4586
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5137	4.1759	0.1080	0.5524	-0.0685	11.0614
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5137	4.1759	0.1080	0.5524	-0.0685	11.0614
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3511	-0.4776	-0.0218	-0.1088	0.0238	-3.7100
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3343	-0.2353	-0.0251	-0.1250	0.0358	-11.2280

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	11.6725
Shear X	-0.8633
Shear Z	0.2900
Moment X	1.5026
Moment Y (Twist)	0.1395
Moment Z	20.7787

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.5034
Shear X	-0.5137
Shear Z	0.1832
Moment X	0.9386
Moment Y (Twist)	0.0828
Moment Z	11.2553

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	-0.0076	2.9193	-0.0074	-0.0380	0.0037	0.1262
ULS: 2. D + L	-0.0076	2.9193	-0.0074	-0.0380	0.0037	0.1262
ULS: 3. D + (S or Lr or R)	-0.0238	8.1155	-0.0233	-0.1205	0.0117	0.3498
ULS: 3. D + (S or Lr or R)	-0.0076	2.9193	-0.0074	-0.0380	0.0037	0.1262
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0198	6.8165	-0.0193	-0.0999	0.0097	0.2939
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0076	2.9193	-0.0074	-0.0380	0.0037	0.1262
ULS: 5b. D + 0.7E	-0.0076	2.9193	-0.0074	-0.0380	0.0037	0.1262

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0198	6.8165	-0.0193	-0.0999	0.0097	0.2939
ULS: 8. 0.6D + 0.7E	-0.0046	1.7516	-0.0044	-0.0228	0.0022	0.0757
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5909	6.4013	-0.0118	-0.0618	-0.0082	12.8129
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5909	6.4013	-0.0118	-0.0618	-0.0082	12.8129
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4148	0.4715	-0.0007	-0.0039	0.0032	-4.2211
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3375	0.7986	-0.0073	-0.0366	0.0184	-12.2462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4572	9.4279	-0.0226	-0.1177	0.0008	9.8089
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4572	9.4279	-0.0226	-0.1177	0.0008	9.8089
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2970	4.9806	-0.0143	-0.0743	0.0094	-2.9666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2391	5.2259	-0.0193	-0.0988	0.0208	-8.9854
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4451	5.5308	-0.0107	-0.0558	-0.0052	9.6412
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4451	5.5308	-0.0107	-0.0558	-0.0052	9.6412
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3092	1.0835	-0.0024	-0.0124	0.0034	-3.1342
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2512	1.3288	-0.0073	-0.0370	0.0148	-9.1531
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5879	5.2335	-0.0088	-0.0466	-0.0097	12.7624
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5879	5.2335	-0.0088	-0.0466	-0.0097	12.7624
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4178	-0.6962	0.0022	0.0113	0.0018	-4.2716
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3406	-0.3691	-0.0044	-0.0214	0.0169	-12.2967

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	14.7155
Shear X	-0.9798
Shear Z	-0.0371
Moment X	-0.1917
Moment Y (Twist)	0.0381
Moment Z	23.8395

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	9.4279
Shear X	-0.5909
Shear Z	-0.0233
Moment X	-0.1205
Moment Y (Twist)	0.0208
Moment Z	12.8129

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	0.0011	2.8270	-0.0000	-0.0000	0.0000	0.0362
ULS: 2. D + L	0.0011	2.8270	-0.0000	-0.0000	0.0000	0.0362
ULS: 3. D + (S or Lr or R)	0.0033	7.8238	-0.0000	-0.0000	0.0000	0.0646
ULS: 3. D + (S or Lr or R)	0.0011	2.8270	-0.0000	-0.0000	0.0000	0.0362
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0028	6.5746	-0.0000	-0.0000	0.0000	0.0575
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0011	2.8270	-0.0000	-0.0000	0.0000	0.0362
ULS: 5b. D + 0.7E	0.0011	2.8270	-0.0000	-0.0000	0.0000	0.0362
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0028	6.5746	-0.0000	-0.0000	0.0000	0.0575
ULS: 8. 0.6D + 0.7E	0.0006	1.6962	-0.0000	-0.0000	0.0000	0.0217
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5826	6.1898	-0.0000	-0.0000	0.0000	12.8759
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5826	6.1898	-0.0000	-0.0000	0.0000	12.8759
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4165	0.4731	-0.0000	-0.0000	0.0000	-4.2709
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3513	0.7707	-0.0000	-0.0000	0.0000	-12.5686
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4350	9.0967	-0.0000	-0.0000	0.0000	9.6872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4350	9.0967	-0.0000	-0.0000	0.0000	9.6872
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3143	4.8092	-0.0000	-0.0000	0.0000	-3.1729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2655	5.0324	-0.0000	-0.0000	0.0000	-9.3962

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	-0.4367	5.3491	-0.0000	-0.0000	0.0000	9.6660
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4367	5.3491	-0.0000	-0.0000	0.0000	9.6660
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3127	1.0616	-0.0000	-0.0000	0.0000	-3.1941
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2638	1.2848	-0.0000	-0.0000	0.0000	-9.4174
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5831	5.0590	-0.0000	-0.0000	0.0000	12.8614
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5831	5.0590	-0.0000	-0.0000	0.0000	12.8614
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4161	-0.6577	-0.0000	-0.0000	0.0000	-4.2854
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3509	-0.3601	-0.0000	-0.0000	0.0000	-12.5831

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	14.1931
Shear X	-0.9728
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	23.9777

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	9.0967
Shear X	-0.5831
Shear Z	-0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	12.8759

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	-0.0076	2.9193	0.0074	0.0380	-0.0037	0.1262
ULS: 2. D + L	-0.0076	2.9193	0.0074	0.0380	-0.0037	0.1262
ULS: 3. D + (S or Lr or R)	-0.0238	8.1155	0.0233	0.1205	-0.0117	0.3498
ULS: 3. D + (S or Lr or R)	-0.0076	2.9193	0.0074	0.0380	-0.0037	0.1262
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0198	6.8165	0.0193	0.0999	-0.0097	0.2939
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0076	2.9193	0.0074	0.0380	-0.0037	0.1262
ULS: 5b. D + 0.7E	-0.0076	2.9193	0.0074	0.0380	-0.0037	0.1262
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0198	6.8165	0.0193	0.0999	-0.0097	0.2939
ULS: 8. 0.6D + 0.7E	-0.0046	1.7516	0.0044	0.0228	-0.0022	0.0757
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5909	6.4013	0.0118	0.0618	0.0082	12.8129
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5909	6.4013	0.0118	0.0618	0.0082	12.8129
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.4148	0.4715	0.0007	0.0039	-0.0032	-4.2211
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3375	0.7986	0.0073	0.0366	-0.0184	-12.2462
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4572	9.4279	0.0226	0.1177	-0.0007	9.8089
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4572	9.4279	0.0226	0.1177	-0.0007	9.8089
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2970	4.9806	0.0143	0.0743	-0.0093	-2.9666
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2391	5.2259	0.0193	0.0988	-0.0207	-8.9854
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4451	5.5308	0.0107	0.0558	0.0052	9.6412
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4451	5.5308	0.0107	0.0558	0.0052	9.6412
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3092	1.0835	0.0024	0.0124	-0.0034	-3.1342
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2512	1.3288	0.0073	0.0370	-0.0147	-9.1531
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5879	5.2335	0.0088	0.0466	0.0097	12.7624
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5879	5.2335	0.0088	0.0466	0.0097	12.7624
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.4178	-0.6962	-0.0022	-0.0113	-0.0017	-4.2716
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3406	-0.3691	0.0044	0.0214	-0.0169	-12.2967

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	14.7155
Shear X	-0.9798
Shear Z	0.0371
Moment X	0.1918
Moment Y (Twist)	0.0381
Moment Z	23.8396

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	9.4279
Shear X	-0.5909
Shear Z	0.0233
Moment X	0.1205
Moment Y (Twist)	0.0207
Moment Z	12.8129

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0071	2.3974	-0.0470	-0.2406	0.0144	-0.0685
ULS: 2. D + L	0.0071	2.3974	-0.0470	-0.2406	0.0144	-0.0685
ULS: 3. D + (S or Lr or R)	0.0222	6.4680	-0.1488	-0.7632	0.0457	-0.2752
ULS: 3. D + (S or Lr or R)	0.0071	2.3974	-0.0470	-0.2406	0.0144	-0.0685
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0184	5.4503	-0.1233	-0.6326	0.0379	-0.2235
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0071	2.3974	-0.0470	-0.2406	0.0144	-0.0685
ULS: 5b. D + 0.7E	0.0071	2.3974	-0.0470	-0.2406	0.0144	-0.0685
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0184	5.4503	-0.1233	-0.6326	0.0379	-0.2235
ULS: 8. 0.6D + 0.7E	0.0043	1.4384	-0.0282	-0.1443	0.0086	-0.0411
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.5109	5.1349	-0.1268	-0.6486	0.0743	11.0341
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.5109	5.1349	-0.1268	-0.6486	0.0743	11.0341
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3539	0.4814	0.0030	0.0126	-0.0180	-3.7374
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.3372	0.7237	0.0063	0.0288	-0.0300	-11.2553
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3701	7.5034	-0.1832	-0.9386	0.0828	8.1034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3701	7.5034	-0.1832	-0.9386	0.0828	8.1034
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2785	4.0133	-0.0858	-0.4427	0.0136	-2.9752
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2660	4.1950	-0.0834	-0.4305	0.0046	-8.6137
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3814	4.4505	-0.1069	-0.5466	0.0593	8.2584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3814	4.4505	-0.1069	-0.5466	0.0593	8.2584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.2672	0.9604	-0.0095	-0.0507	-0.0099	-2.8202
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.2547	1.1421	-0.0071	-0.0385	-0.0189	-8.4586
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.5137	4.1759	-0.1080	-0.5524	0.0685	11.0614
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.5137	4.1759	-0.1080	-0.5524	0.0685	11.0614
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3511	-0.4776	0.0218	0.1088	-0.0238	-3.7100
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.3343	-0.2353	0.0251	0.1250	-0.0358	-11.2279

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	11.6725
Shear X	-0.8633
Shear Z	-0.2900
Moment X	-1.5026
Moment Y (Twist)	0.1396
Moment Z	20.7790

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	7.5034
Shear X	-0.5137
Shear Z	-0.1832
Moment X	-0.9386
Moment Y (Twist)	0.0828
Moment Z	11.2553

Project Details

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

User Name: sales@mtsolar.us
 Project Name: D50Schools-Manor-JB-Area1
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
7	6in Pipe Sch 40	6.63	0.28				

ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	

ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I _{yp} (in ⁴)	I _{zp} (in ⁴)	I _w (in ⁶)	S _{yp} (in ³)	S _{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

412	5	1.30	1.30	2.00	-	300	200	1
413	19	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.10,1.10,1.13,1.46,1.10,1.10,1.06,1.19,1.11,1.11,1.12,1.12,1.10,1.10,1.13,1.55,1.10,1.10,1.07,1.15	300	200	1
414	19	4.88	4.00	7.50	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.04,1.13,1.11,1.11,1.13,1.24,1.11,1.11,1.11,1.12,1.11,1.11,1.06,1.13,1.11,1.11,1.13,1.34	300	200	1
415	19	8.42	8.42	12.95	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.21,1.12,1.12,1.11,1.14,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.24,1.12,1.12,1.11,1.14	300	200	1
416	19	8.42	8.42	12.95	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.15,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.17	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	40.96	42.30	42.30	75.35	75.35
2	198.33	196.72	21.95	21.95	59.50	59.50
3	116.10	115.41	15.79	11.10	42.08	23.28
4	116.10	111.33	15.79	11.10	42.08	23.28
5	116.10	114.23	15.79	11.10	42.08	23.28
6	116.10	115.41	15.79	11.10	42.08	23.28
7	116.10	114.23	15.79	11.10	42.08	23.28
8	133.20	126.01	32.87	6.12	40.24	43.62
9	66.48	58.89	3.82	3.82	19.94	19.94
10	116.10	111.33	15.79	11.10	42.08	23.28
11	133.20	126.01	32.87	6.12	40.24	43.62
12	198.33	196.72	21.95	21.95	59.50	59.50
13	133.20	104.94	24.29	6.12	40.24	43.62
14	133.20	104.94	23.83	6.12	40.24	43.62
15	133.20	52.83	32.87	6.12	40.24	43.62
16	133.20	52.83	32.87	6.12	40.24	43.62
101	251.16	40.96	42.30	42.30	75.35	75.35
102	198.33	196.72	21.95	21.95	59.50	59.50
103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	104.94	23.60	6.12	40.24	43.62
114	133.20	104.94	23.37	6.12	40.24	43.62
115	133.20	46.28	12.02	6.12	40.24	43.62
116	133.20	46.28	12.13	6.12	40.24	43.62
201	251.16	40.96	42.30	42.30	75.35	75.35
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28

208	133.20	126.01	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	126.01	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	104.94	23.60	6.12	40.24	43.62
214	133.20	104.94	23.37	6.12	40.24	43.62
215	133.20	46.28	12.35	6.12	40.24	43.62
216	133.20	46.28	12.46	6.12	40.24	43.62
301	251.16	40.96	42.30	42.30	75.35	75.35
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	126.01	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	126.01	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	104.94	23.60	6.12	40.24	43.62
314	133.20	104.94	23.37	6.12	40.24	43.62
315	133.20	46.28	12.13	6.12	40.24	43.62
316	133.20	46.28	12.46	6.12	40.24	43.62
401	251.16	40.96	42.30	42.30	75.35	75.35
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28
408	133.20	52.83	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	52.83	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	104.94	24.29	6.12	40.24	43.62
414	133.20	104.94	23.83	6.12	40.24	43.62
415	133.20	46.28	12.02	6.12	40.24	43.62
416	133.20	46.28	12.02	6.12	40.24	43.62

Design Ratio

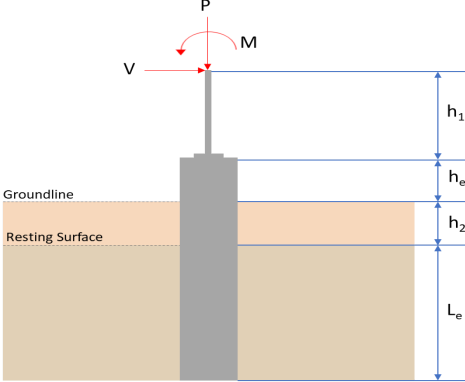
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.285	0.491	0.071	0.011	0.004	0.685	#13	0.877	Not Required	Pass
2	0.002	0.383	0.040	0.085	0.006	0.417	#21	0.035	Not Required	Pass
3	0.003	0.617	0.016	0.061	0.006	0.622	#21	0.045	Not Required	Pass
4	0.002	0.568	0.053	0.057	0.012	0.613	#21	0.080	Not Required	Pass
5	0.003	0.382	0.042	0.062	0.011	0.389	#21	0.074	Not Required	Pass
6	0.004	0.763	0.038	0.078	0.008	0.803	#21	0.045	Not Required	Pass
7	0.004	0.473	0.094	0.076	0.024	0.495	#21	0.074	Not Required	Pass
8	0.001	0.110	0.101	0.049	0.007	0.140	#21	0.095	Not Required	Pass

9	0.006	0.092	0.034	0.003	0.002	0.128	#21	0.204	Not Required	Pass
10	0.005	0.685	0.081	0.069	0.016	0.713	#21	0.080	Not Required	Pass
11	0.002	0.110	0.104	0.055	0.007	0.137	#21	0.095	Not Required	Pass
12	0.001	0.521	0.047	0.105	0.009	0.560	#21	0.053	Not Required	Pass
13	0.003	0.217	0.209	0.069	0.009	0.331	#21	0.286	Not Required	Pass
14	0.003	0.196	0.205	0.062	0.009	0.339	#24	0.190	Not Required	Pass
15	0.000	0.067	0.048	0.029	0.004	0.115	#21	Not Required	Not Required	Pass
16	0.000	0.062	0.048	0.027	0.004	0.110	#21	Not Required	Not Required	Pass
101	0.359	0.564	0.009	0.013	0.000	0.794	#13	0.877	Not Required	Pass
102	0.002	0.589	0.054	0.122	0.007	0.638	#21	0.035	Not Required	Pass
103	0.004	0.874	0.024	0.088	0.001	0.900	#21	0.045	Not Required	Pass
104	0.005	0.819	0.097	0.082	0.020	0.879	#21	0.080	Not Required	Pass
105	0.005	0.542	0.103	0.087	0.026	0.569	#21	0.074	Not Required	Pass
106	0.004	0.863	0.023	0.087	0.004	0.879	#21	0.045	Not Required	Pass
107	0.004	0.536	0.087	0.086	0.022	0.561	#21	0.074	Not Required	Pass
108	0.002	0.092	0.096	0.055	0.007	0.181	#21	0.095	Not Required	Pass
109	0.010	0.094	0.027	0.001	0.001	0.126	#21	0.204	Not Required	Pass
110	0.004	0.796	0.086	0.080	0.018	0.849	#21	0.080	Not Required	Pass
111	0.003	0.090	0.098	0.059	0.007	0.180	#21	0.095	Not Required	Pass
112	0.002	0.572	0.055	0.119	0.010	0.621	#21	0.035	Not Required	Pass
113	0.003	0.341	0.232	0.076	0.010	0.541	#21	0.286	Not Required	Pass
114	0.005	0.340	0.231	0.071	0.010	0.544	#21	0.286	Not Required	Pass
115	0.007	0.593	0.124	0.062	0.008	0.701	#21	0.601	Not Required	Pass
116	0.004	0.541	0.123	0.059	0.008	0.649	#21	0.601	Not Required	Pass
201	0.346	0.567	0.000	0.013	0.000	0.785	#13	0.877	Not Required	Pass
202	0.002	0.558	0.053	0.116	0.008	0.605	#21	0.035	Not Required	Pass
203	0.004	0.844	0.021	0.085	0.001	0.867	#21	0.045	Not Required	Pass
204	0.004	0.770	0.080	0.077	0.017	0.822	#21	0.080	Not Required	Pass
205	0.004	0.524	0.084	0.084	0.021	0.547	#21	0.074	Not Required	Pass
206	0.004	0.844	0.021	0.085	0.001	0.867	#21	0.045	Not Required	Pass
207	0.004	0.524	0.084	0.084	0.021	0.547	#21	0.074	Not Required	Pass
208	0.002	0.074	0.087	0.053	0.007	0.162	#21	0.095	Not Required	Pass
209	0.008	0.089	0.021	0.001	0.000	0.114	#21	0.204	Not Required	Pass
210	0.004	0.770	0.080	0.077	0.017	0.822	#21	0.080	Not Required	Pass
211	0.003	0.081	0.089	0.058	0.007	0.170	#21	0.095	Not Required	Pass
212	0.002	0.558	0.053	0.116	0.008	0.605	#21	0.035	Not Required	Pass
213	0.003	0.326	0.189	0.072	0.009	0.497	#21	0.286	Not Required	Pass
214	0.005	0.306	0.188	0.066	0.009	0.469	#21	0.286	Not Required	Pass
215	0.007	0.470	0.103	0.058	0.007	0.575	#21	0.601	Not Required	Pass
216	0.006	0.424	0.104	0.053	0.007	0.526	#21	0.601	Not Required	Pass
301	0.359	0.564	0.009	0.013	0.000	0.794	#13	0.877	Not Required	Pass
302	0.002	0.572	0.055	0.119	0.010	0.621	#21	0.035	Not Required	Pass
303	0.004	0.863	0.023	0.087	0.004	0.879	#21	0.045	Not Required	Pass
304	0.004	0.796	0.086	0.080	0.018	0.849	#21	0.080	Not Required	Pass
305	0.004	0.536	0.087	0.086	0.022	0.561	#21	0.074	Not Required	Pass
306	0.004	0.874	0.024	0.088	0.001	0.900	#21	0.045	Not Required	Pass
307	0.005	0.542	0.103	0.087	0.026	0.569	#21	0.074	Not Required	Pass
308	0.001	0.085	0.123	0.059	0.008	0.208	#21	0.095	Not Required	Pass
309	0.010	0.094	0.027	0.001	0.001	0.126	#21	0.204	Not Required	Pass
310	0.005	0.819	0.097	0.082	0.020	0.879	#21	0.080	Not Required	Pass
311	0.002	0.075	0.124	0.062	0.008	0.200	#21	0.095	Not Required	Pass
312	0.002	0.589	0.054	0.122	0.007	0.638	#21	0.035	Not Required	Pass
313	0.003	0.341	0.233	0.076	0.010	0.541	#21	0.286	Not Required	Pass

314	0.005	0.340	0.231	0.071	0.010	0.544	#21	0.286	Not Required	Pass
315	0.007	0.468	0.105	0.059	0.007	0.573	#21	0.601	Not Required	Pass
316	0.006	0.417	0.104	0.055	0.007	0.523	#21	0.601	Not Required	Pass
401	0.285	0.491	0.071	0.011	0.004	0.685	#13	0.877	Not Required	Pass
402	0.001	0.521	0.047	0.105	0.009	0.560	#21	0.053	Not Required	Pass
403	0.004	0.763	0.038	0.078	0.008	0.803	#21	0.045	Not Required	Pass
404	0.005	0.685	0.081	0.069	0.016	0.713	#21	0.080	Not Required	Pass
405	0.004	0.473	0.094	0.076	0.024	0.495	#21	0.074	Not Required	Pass
406	0.003	0.617	0.016	0.061	0.006	0.622	#21	0.045	Not Required	Pass
407	0.003	0.382	0.042	0.062	0.011	0.389	#21	0.074	Not Required	Pass
408	0.000	0.062	0.048	0.027	0.004	0.110	#21	Not Required	Not Required	Pass
409	0.006	0.092	0.034	0.003	0.002	0.128	#21	0.204	Not Required	Pass
410	0.002	0.568	0.053	0.057	0.012	0.613	#21	0.080	Not Required	Pass
411	0.000	0.067	0.048	0.029	0.004	0.115	#21	Not Required	Not Required	Pass
412	0.002	0.383	0.040	0.085	0.006	0.417	#21	0.035	Not Required	Pass
413	0.003	0.217	0.208	0.069	0.009	0.331	#21	0.190	Not Required	Pass
414	0.003	0.196	0.205	0.062	0.009	0.339	#24	0.286	Not Required	Pass
415	0.007	0.615	0.105	0.055	0.007	0.723	#21	0.601	Not Required	Pass
416	0.004	0.568	0.107	0.049	0.007	0.676	#21	0.601	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 1102 1193 1193"> <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>7.503</td> <td>11.673</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.514</td> <td>-0.863</td> </tr> <tr> <td>V_z (kip)</td> <td>0.183</td> <td>0.290</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.939</td> <td>1.503</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.255</td> <td>20.779</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)	7.503	11.673	V_x (kip)	-0.514	-0.863	V_z (kip)	0.183	0.290	M_x (kipft)	0.939	1.503	M_z (kipft)	11.255	20.779	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.514 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.081847 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.255 \text{ kipft}) + ((-0.514 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7922 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.9215 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.183 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.02914 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((0.183 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14952 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.5409 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9215 \text{ ft}), (2.5409 \text{ ft})]$$

$$L_{e,req} = 4.922 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.922 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.93752$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.503 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46894 \text{ kip/ft}^2$$

$$q = 0.46894 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46894 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23447$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.081847 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7922 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7922 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.7922 \text{ kipft/ft})) + (4 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5603 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.7922 \text{ kipft/ft})) + (3 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.7922 \text{ kipft/ft})) + (2 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20825 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.7922 \text{ kipft/ft})) + ((-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.68674 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5603 \text{ ft})}{2}$$

$$p_a = 0.26702 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20825 \text{ kip/ft}^2)}{(0.26702 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7799$$

p_a - Allowable lateral soil pressure at depth L_e ,

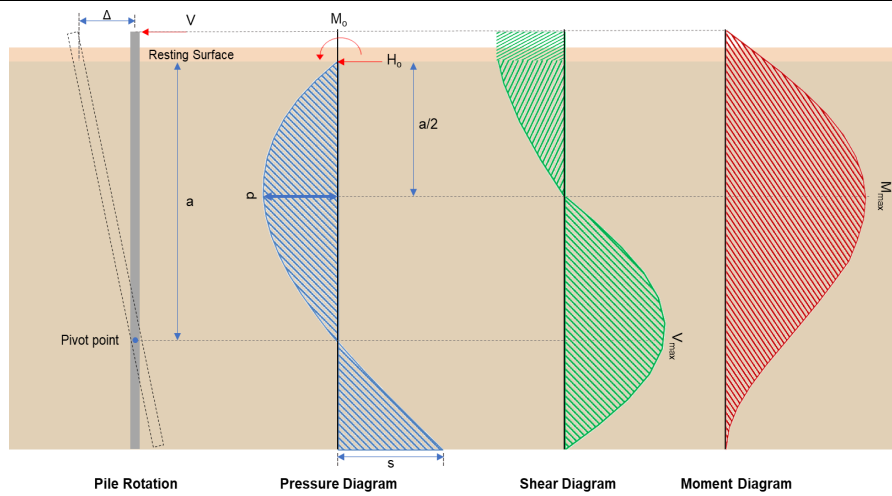
Status: **PASS**
Ratio: **0.780**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.68674 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.87205$	Status: PASS Ratio: 0.870
	<p>Considering z-direction:</p> <p>$H_o = 0.02914 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.14952 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.02914 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.6774 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.14952 \text{ kipft/ft})) + (3 \times (0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.14952 \text{ kipft/ft})) + (2 \times (0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = 0.040295 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.14952 \text{ kipft/ft})) + ((0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = 0.098401 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.6774 \text{ ft})}{2}$ $p_a = 0.27581 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.040295 \text{ kip/ft}^2)}{(0.27581 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.1461$ <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: 0.150

$$\text{Ratio} = \frac{(0.098401 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.12495$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.863 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13742 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.779 \text{ kipft}) + ((-0.863 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3088 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3088 \text{ kipft/ft})}{(-0.13742 \text{ kip/ft})}$$

$$E = 24.078 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3088 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.13742 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3088 \text{ kipft/ft})) + (4 \times (-0.13742 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5555 \text{ ft}$$

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

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

$$V_{max} = ((-0.13742 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5555 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5555 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.8317 \text{ kip}$$

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

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

$$M_{max} = ((-0.13742 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(24.078 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.418 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.29 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.503 \text{ kipft}) + ((0.29 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23933 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23933 \text{ kipft/ft})}{(0.046178 \text{ kip/ft})}$$

$$E = 5.1828 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23933 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.046178 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.23933 \text{ kipft/ft})) + (4 \times (0.046178 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6764 \text{ ft}$$

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

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

$$V_{max} = ((0.046178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6764 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6764 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44468 \text{ kip}$$

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

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

$$M_{max} = ((0.046178 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.1828 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0799 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.673 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.208 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.208 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(11.673 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0043634$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.673 \text{ kip} \rightarrow 11673 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11673 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.04 \text{ kip}$$

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

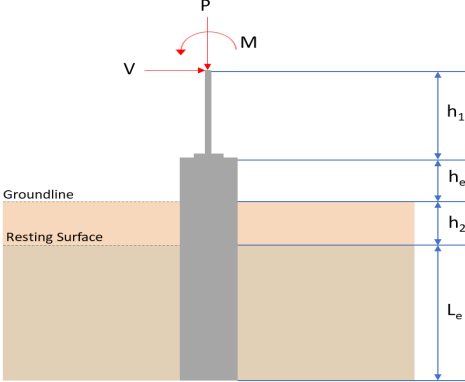
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.04 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.04 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.04 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.11 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.8317 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.8317 \text{ kip})}{(111.11 \text{ kip})}$ $\text{Ratio} = 0.043486$ <p>Considering z-direction:</p> <p>$V_{max} = 0.44468 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.44468 \text{ kip})}{(111.11 \text{ kip})}$ $\text{Ratio} = 0.0040022$	<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: $M_{max} = 12.418 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.418 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.049753$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 1.0799 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.0799 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0043267$	<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 1193"> <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>9.428</td> <td>14.716</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.591</td> <td>-0.980</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.023</td> <td>-0.037</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.120</td> <td>-0.192</td> </tr> <tr> <td>M_z (kipft)</td> <td>12.813</td> <td>23.839</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)	9.428	14.716	V_x (kip)	-0.591	-0.980	V_z (kip)	-0.023	-0.037	M_x (kipft)	-0.120	-0.192	M_z (kipft)	12.813	23.839	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.591 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.094108 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(12.813 \text{ kipft}) + ((-0.591 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.0403 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.1211 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.023 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0036624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.12 \text{ kipft}) + ((-0.023 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.019108 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0885 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1211 \text{ ft}), (1.0885 \text{ ft})]$$

$$L_{e,req} = 5.121 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.121 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.97543$$

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

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

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

$$q = \frac{(9.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58925 \text{ kip/ft}^2$$

$$q = 0.00920 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58925 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29463$$

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:

$H_o = -0.094108 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.0403 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0403 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.0403 \text{ kipft/ft})) + (4 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5608 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.0403 \text{ kipft/ft})) + (3 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (2.0403 \text{ kipft/ft})) + (2 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.23649 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.0403 \text{ kipft/ft})) + ((-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.78074 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5608 \text{ ft})}{2}$$

$$p_a = 0.26706 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23649 \text{ kip/ft}^2)}{(0.26706 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88552$$

p_a - Allowable lateral soil pressure at depth L_e ,

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.78074 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99141$$

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

Considering z-direction:

$H_o = -0.0036624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.019108 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019108 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.019108 \text{ kipft/ft})) + (4 \times (-0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6757 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.019108 \text{ kipft/ft})) + (3 \times (-0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.019108 \text{ kipft/ft})) + (2 \times (-0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.00050697 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.019108 \text{ kipft/ft})) + ((-0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.0041336 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6757 \text{ ft})}{2}$$

$$p_a = 0.27567 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00050697 \text{ kip/ft}^2)}{(0.27567 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.001839$$

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

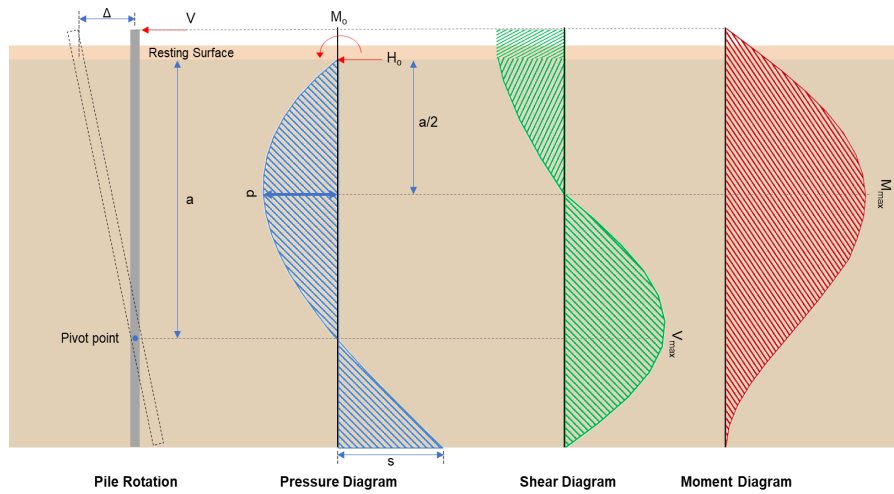
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0041336 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0052491$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.98 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(23.839 \text{ kipft}) + ((-0.98 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.796 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.796 \text{ kipft/ft})}{(-0.15605 \text{ kip/ft})}$$

$$E = 24.326 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.796 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.15605 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.796 \text{ kipft/ft})) + (4 \times (-0.15605 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.555 \text{ ft}$$

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

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

$$V_{max} = ((-0.15605 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.326 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.555 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.326 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.555 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.5391 \text{ kip}$$

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

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

$$M_{max} = ((-0.15605 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(24.326 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.326 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (24.326 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.239 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0058917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.192 \text{ kipft}) + ((-0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.030573 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.030573 \text{ kipft/ft})}{(-0.0058917 \text{ kip/ft})}$$

$$E = 5.1892 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.030573 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.030573 \text{ kipft/ft})) + (4 \times (-0.0058917 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6762 \text{ ft}$$

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

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

$$V_{max} = ((-0.0058917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6762 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6762 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.056786 \text{ kip}$$

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

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

$$M_{max} = ((-0.0058917 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.1892 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.13792 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.716 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.107 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.107 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(14.716 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0055009$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.716 \text{ kip} \rightarrow 14716 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14716 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.45 \text{ kip}$$

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

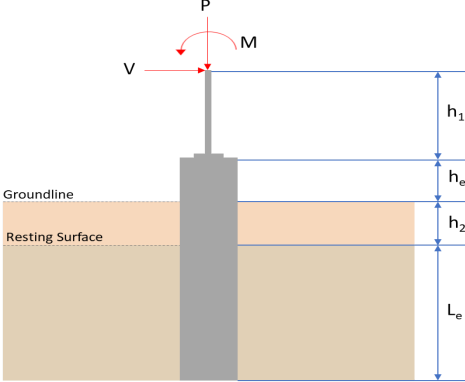
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.45 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.45 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{ytik} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.45 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.37 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 5.5391 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(5.5391 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.049735$ <p>Considering z-direction:</p> <p>$V_{max} = 0.056786 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.056786 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.00050988$	<p>Status: PASS Ratio: 0.050</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: $M_{max} = 14.239 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.239 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.057049$	<p>Status: PASS Ratio: 0.060</p>
	<p>Considering z-direction: $M_{max} = 0.13792 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.13792 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.00055256$	<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 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.097</td> <td>14.193</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.583</td> <td>-0.973</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>12.876</td> <td>23.978</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)	9.097	14.193	V_x (kip)	-0.583	-0.973	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	12.876	23.978	
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																										
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V_x (kip)	-0.583	-0.973																										
V_z (kip)	0.000	0.000																										
M_x (kipft)	0.000	0.000																										
M_z (kipft)	12.876	23.978																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.583 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.092834 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(12.876 \text{ kipft}) + ((-0.583 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.0503 \text{ kipft/ft}$$

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:

$$L_{e,x} = 5.1353 \text{ ft} - \text{Required depth in x-direction,}$$

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1353 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 5.135 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.135 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.9781$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(9.097 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.56856 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.56856 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.28428$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.092834$ kip/ft - Lateral force per length of pile,

$M_o = 2.0503$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0503 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.092834 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.0503 \text{ kipft/ft})) + (4 \times (-0.092834 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5598 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.0503 \text{ kipft/ft})) + (3 \times (-0.092834 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (2.0503 \text{ kipft/ft})) + (2 \times (-0.092834 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.23875 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.0503 \text{ kipft/ft})) + ((-0.092834 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.78656 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5598 \text{ ft})}{2}$$

$$p_a = 0.26699 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23875 \text{ kip/ft}^2)}{(0.26699 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89423$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

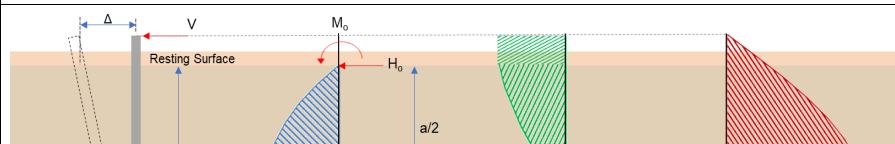
$$\text{Ratio} = \frac{s}{p_s}$$

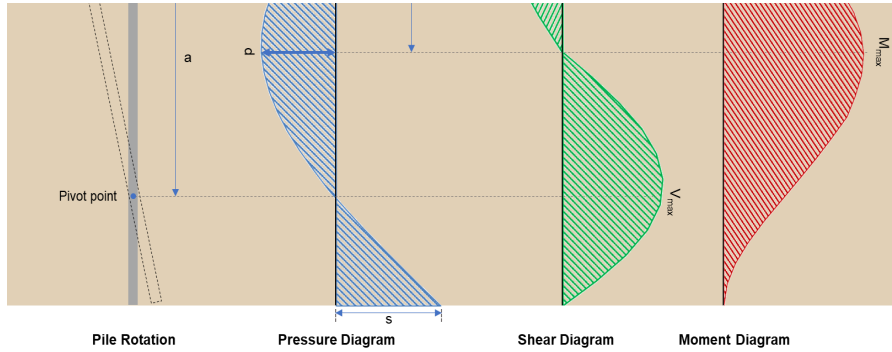
$$\text{Ratio} = \frac{(0.78656 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99881$$

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

Status: **PASS**
Ratio: **1.000**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.973 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15494 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(23.978 \text{ kipft}) + ((-0.973 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.8182 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.8182 \text{ kipft/ft})}{(-0.15494 \text{ kip/ft})}$$

$$E = 24.643 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.8182 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.15494 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.8182 \text{ kipft/ft})) + (4 \times (-0.15494 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5544 \text{ ft}$$

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

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

$$V_{max} = ((-0.15494 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.643 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5544 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.643 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5544 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.5662 \text{ kip}$$

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

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

$$M_{max} = ((-0.15494 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(24.643 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5544 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.643 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5544 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (24.643 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5544 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.313 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.193 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.125 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.125 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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)**

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

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

$$\text{Ratio} = \frac{(14.193 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0053054$$

Status: **PASS**
Ratio: **0.010****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.193 \text{ kip} \rightarrow 14193 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14193 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.38 \text{ kip}$$

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

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

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

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

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.38 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.38 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.38 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.33 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 5.5662 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(5.5662 \text{ kip})}{(111.33 \text{ kip})}$ $\text{Ratio} = 0.05$	<p>Status: PASS Ratio: 0.050</p>
<p>14.5.2.1b</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>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</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> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

$\phi M_{n,2} = \phi M_{n,1}$

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

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

Therefore,
 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

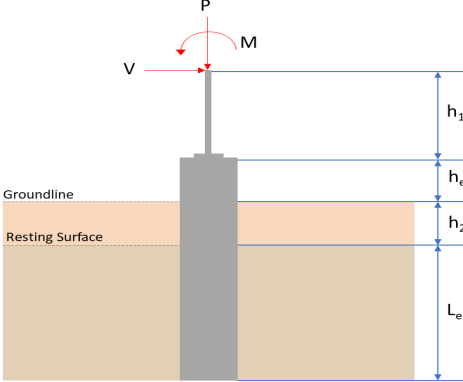
$M_{max} = 14.313 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(14.313 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.057343$$

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

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>9.428</td> <td>14.716</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.591</td> <td>-0.980</td> </tr> <tr> <td>V_z (kip)</td> <td>0.023</td> <td>0.037</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.120</td> <td>0.192</td> </tr> <tr> <td>M_z (kipft)</td> <td>12.813</td> <td>23.840</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)	9.428	14.716	V_x (kip)	-0.591	-0.980	V_z (kip)	0.023	0.037	M_x (kipft)	0.120	0.192	M_z (kipft)	12.813	23.840	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.591 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.094108 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(12.813 \text{ kipft}) + ((-0.591 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.0403 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.1211 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.023 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0036624 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.12 \text{ kipft}) + ((0.023 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.019108 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2155 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.1211 \text{ ft}), (1.2155 \text{ ft})]$$

$$L_{e,req} = 5.121 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.121 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.97543$$

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

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

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

$$q = \frac{(9.428 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.58925 \text{ kip/ft}^2$$

$$q = 0.00920 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.58925 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.29463$$

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:

$H_o = -0.094108 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.0403 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.0403 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (2.0403 \text{ kipft/ft})) + (4 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5608 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (2.0403 \text{ kipft/ft})) + (3 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (2.0403 \text{ kipft/ft})) + (2 \times (-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.23649 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (2.0403 \text{ kipft/ft})) + ((-0.094108 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.78074 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5608 \text{ ft})}{2}$$

$$p_a = 0.26706 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23649 \text{ kip/ft}^2)}{(0.26706 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88552$$

p_a - Allowable lateral soil pressure at depth L_e ,

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.78074 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99141$$

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

Considering z-direction:

$H_o = 0.0036624 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.019108 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019108 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.019108 \text{ kipft/ft})) + (4 \times (0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6757 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.019108 \text{ kipft/ft})) + (3 \times (0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.019108 \text{ kipft/ft})) + (2 \times (0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.0051101 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.019108 \text{ kipft/ft})) + ((0.0036624 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.012505 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.6757 \text{ ft})}{2}$$

$$p_a = 0.27567 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0051101 \text{ kip/ft}^2)}{(0.27567 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.018537$$

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

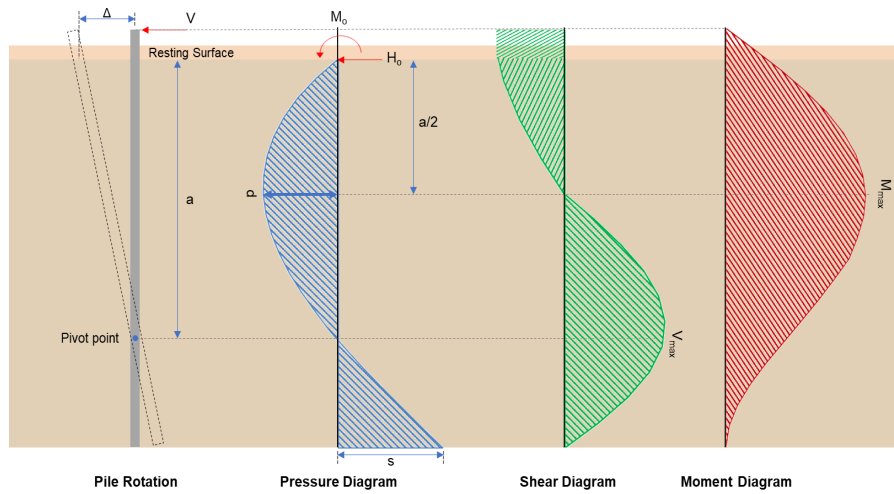
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.012505 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.015879$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.98 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(23.84 \text{ kipft}) + ((-0.98 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.7962 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.7962 \text{ kipft/ft})}{(-0.15605 \text{ kip/ft})}$$

$$E = 24.327 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.7962 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.15605 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.7962 \text{ kipft/ft})) + (4 \times (-0.15605 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.555 \text{ ft}$$

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

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

$$V_{max} = ((-0.15605 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.327 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.555 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.327 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.555 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.5393 \text{ kip}$$

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

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

$$M_{max} = ((-0.15605 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(24.327 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.327 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (24.327 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 14.24 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0058917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.192 \text{ kipft}) + ((0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.030573 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.030573 \text{ kipft/ft})}{(0.0058917 \text{ kip/ft})}$$

$$E = 5.1892 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.030573 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.030573 \text{ kipft/ft})) + (4 \times (0.0058917 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6762 \text{ ft}$$

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

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

$$V_{max} = ((0.0058917 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6762 \text{ ft})}{(5.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6762 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.056786 \text{ kip}$$

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

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

$$M_{max} = ((0.0058917 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.1892 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (5.1892 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6762 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.13792 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(14.716 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.107 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.107 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(14.716 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0055009$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 14.716 \text{ kip} \rightarrow 14716 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(14716 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.45 \text{ kip}$$

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

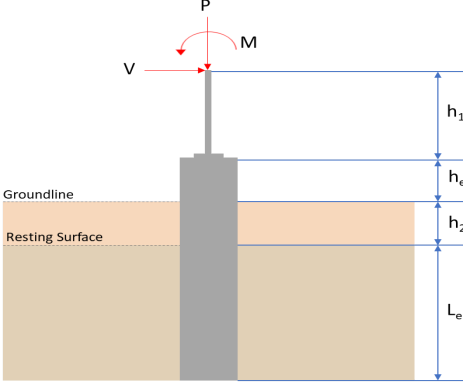
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.45 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.45 \text{ kip}$$

<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> $V_{s,b} = \frac{2 A_v f_{ytks} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.45 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.37 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 5.5393 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(5.5393 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.049737$ <p>Considering z-direction:</p> <p>$V_{max} = 0.056786 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.056786 \text{ kip})}{(111.37 \text{ kip})}$ $\text{Ratio} = 0.00050988$	<p>Status: PASS Ratio: 0.050</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: $M_{max} = 14.24 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(14.24 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.057051$	<p>Status: PASS Ratio: 0.060</p>
	<p>Considering z-direction: $M_{max} = 0.13792 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.13792 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00055256$	<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>7.503</td> <td>11.672</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.514</td> <td>-0.863</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.183</td> <td>-0.290</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.939</td> <td>-1.503</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.255</td> <td>20.779</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)	7.503	11.672	V_x (kip)	-0.514	-0.863	V_z (kip)	-0.183	-0.290	M_x (kipft)	-0.939	-1.503	M_z (kipft)	11.255	20.779	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.514 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.081847 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.255 \text{ kipft}) + ((-0.514 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.7922 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.9215 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.183 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.02914 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((-0.183 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.14952 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0333 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.9215 \text{ ft}), (2.0333 \text{ ft})]$$

$$L_{e,req} = 4.922 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.922 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.93752$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.503 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.46894 \text{ kip/ft}^2$$

$$q = 0.46894 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.46894 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23447$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.3125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.081847 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.7922 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.7922 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.7922 \text{ kipft/ft})) + (4 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5603 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.7922 \text{ kipft/ft})) + (3 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.7922 \text{ kipft/ft})) + (2 \times (-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.20825 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.7922 \text{ kipft/ft})) + ((-0.081847 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.68674 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5603 \text{ ft})}{2}$$

$$p_a = 0.26702 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.20825 \text{ kip/ft}^2)}{(0.26702 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7799$$

p_a - Allowable lateral soil pressure at depth L_e ,

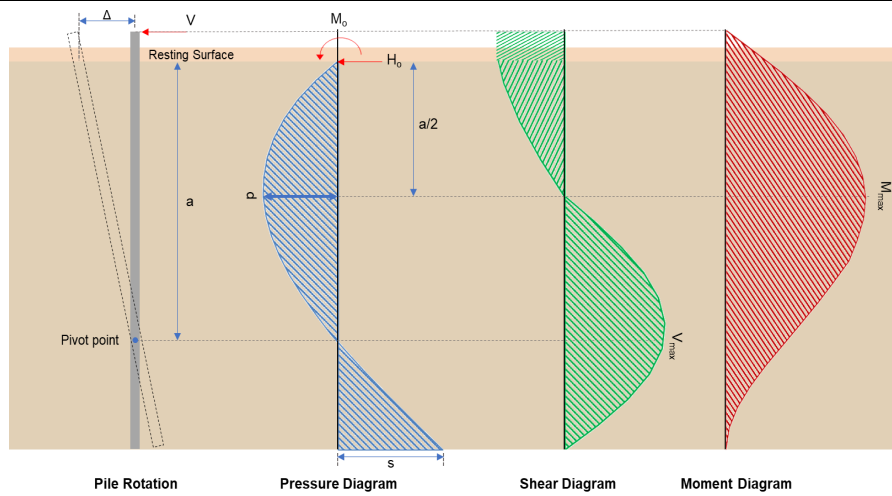
Status: **PASS**
Ratio: **0.780**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.68674 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.87205$	Status: PASS Ratio: 0.870
	<p>Considering z-direction:</p> <p>$H_o = -0.02914 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.14952 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.14952 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.02914 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.14952 \text{ kipft/ft})) + (4 \times (-0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.6774 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.14952 \text{ kipft/ft})) + (3 \times (-0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 \times [(3 \times (0.14952 \text{ kipft/ft})) + (2 \times (-0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = 0.003694 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.14952 \text{ kipft/ft})) + ((-0.02914 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = 0.031795 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.6774 \text{ ft})}{2}$ $p_a = 0.27581 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.003694 \text{ kip/ft}^2)}{(0.27581 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.013393$ <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: 0.010

$$\text{Ratio} = \frac{(0.031795 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.040375$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.863 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13742 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.779 \text{ kipft}) + ((-0.863 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3088 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.3088 \text{ kipft/ft})}{(-0.13742 \text{ kip/ft})}$$

$$E = 24.078 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3088 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.13742 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.3088 \text{ kipft/ft})) + (4 \times (-0.13742 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5555 \text{ ft}$$

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

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

$$V_{max} = ((-0.13742 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5555 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5555 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.8317 \text{ kip}$$

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

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

$$M_{max} = ((-0.13742 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(24.078 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (24.078 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5555 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.418 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.29 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.503 \text{ kipft}) + ((-0.29 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.23933 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.23933 \text{ kipft/ft})}{(-0.046178 \text{ kip/ft})}$$

$$E = 5.1828 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.23933 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.046178 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.23933 \text{ kipft/ft})) + (4 \times (-0.046178 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.6764 \text{ ft}$$

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

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

$$V_{max} = ((-0.046178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6764 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6764 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.44468 \text{ kip}$$

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

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

$$M_{max} = ((-0.046178 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(5.1828 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1828 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.6764 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0799 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.672 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.208 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.208 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(11.672 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0043631$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.672 \text{ kip} \rightarrow 11672 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11672 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.04 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.04 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.04 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.04 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.11 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.8317 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.8317 \text{ kip})}{(111.11 \text{ kip})}$ $\text{Ratio} = 0.043486$ <p>Considering z-direction:</p> <p>$V_{max} = 0.44468 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.44468 \text{ kip})}{(111.11 \text{ kip})}$ $\text{Ratio} = 0.0040022$	<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: $M_{max} = 12.418 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.418 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.049753$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 1.0799 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.0799 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0043267$	<p>Status: PASS Ratio: 0.000</p>