

Your Project Calculations



Project Name: Leidos PV Canopy v2

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Leidos%20PV%20Canopy%20v2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/3_2024

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=MjmhD4EXefzZmDL2tTTOwg3HayPgdD5i33bD3321DLYZIOpcEoO5UbotFIH8JO8B

Array Specification

Product:	Beam
Unique ID:	4P-19.75-6TOP-HD-12-L-5Hx10W-812D
Duty Classification:	HD
Module Width:	40.89 in
Module Length:	82.44in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	5
Front Edge Clearance:	9.5
Total Array Height at Tilt:	10.99 ft
Total Frame Length:	68.75 ft
Frame Weight:	3011 lbs
Array Dimensions N/S:	17.25 ft
Array Dimensions E/W:	69.53 ft
Rail Length:	206.95 in
Rail Spacing:	3.44 ft
Rail Check:	Not Checked

Support Specifications

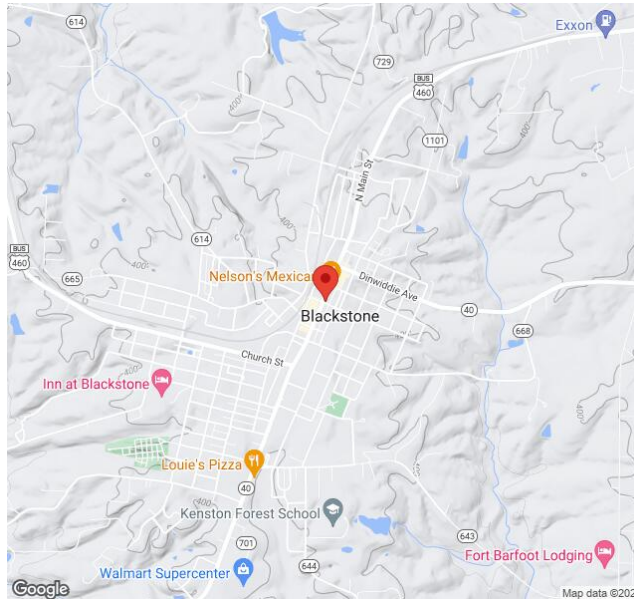
Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	10.25 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.00 ft Pile 2: 5.25 ft Pile 3: 5.25 ft Pile 4: 5.00 ft
Foundation Volume:	12.148 y ³
Foundation Result:	PASSED
Mount Twist:	0.141364 kip

Site Info

Risk Category:	I
Exposure:	D
Soil Classification:	sand
Site Location:	Blackstone, VA 23824, USA
Wind Speed:	105 mph
Snow Load:	20 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.010752 ksf



Design Disclaimer

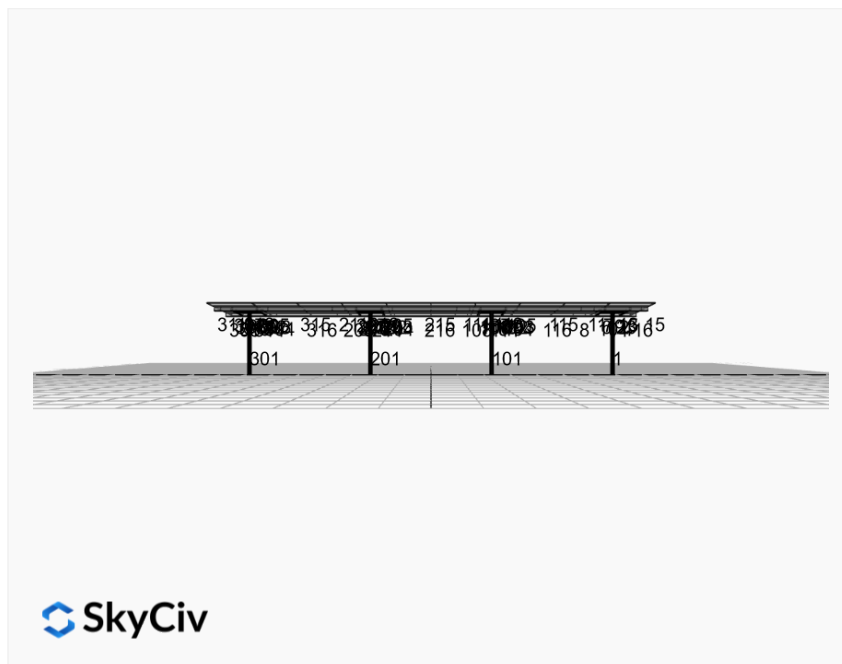
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

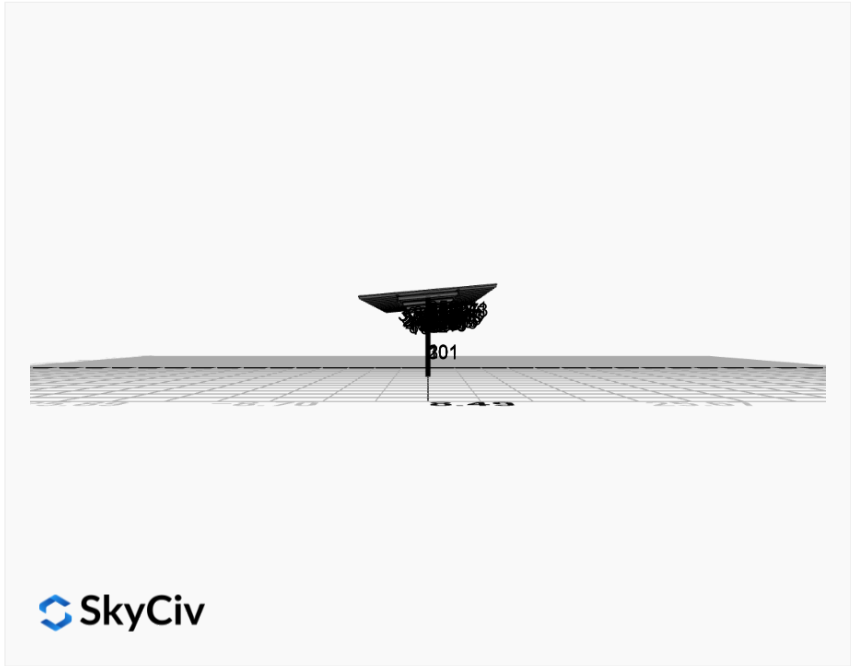
AutoDesigner Input

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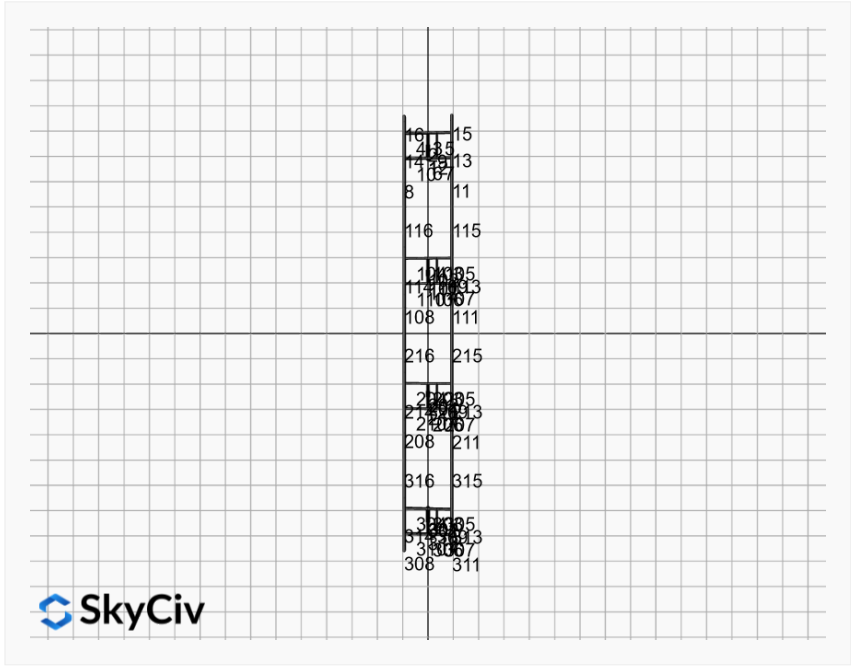
Design Notes:

- 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
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame design checks are based on AISC 360 2016 (LRFD)
- Foundation Design and Sizing is approximate only



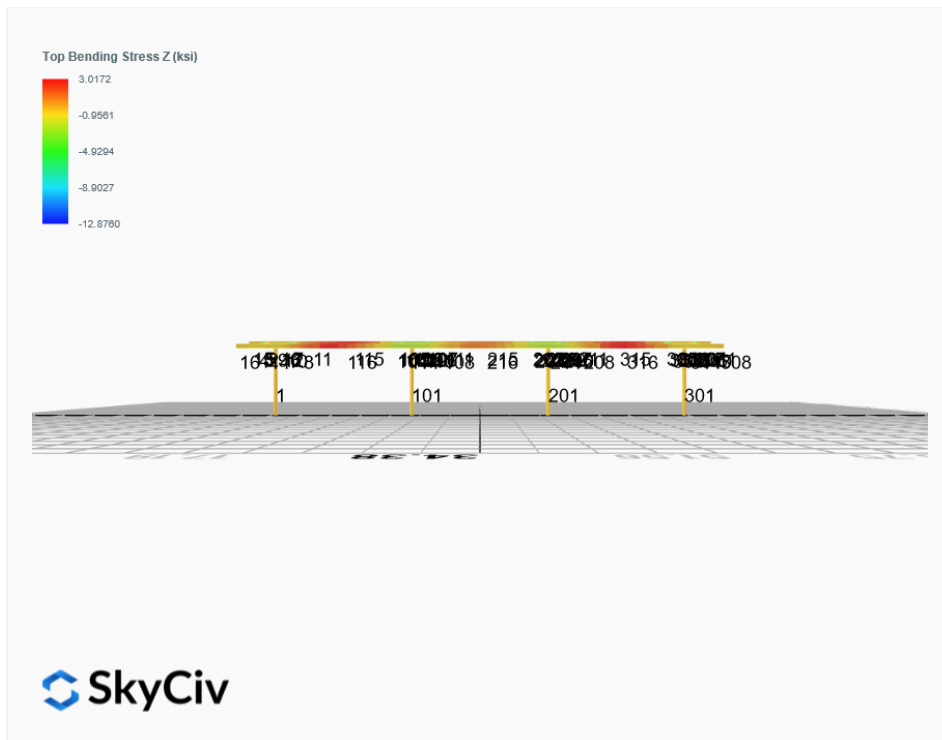
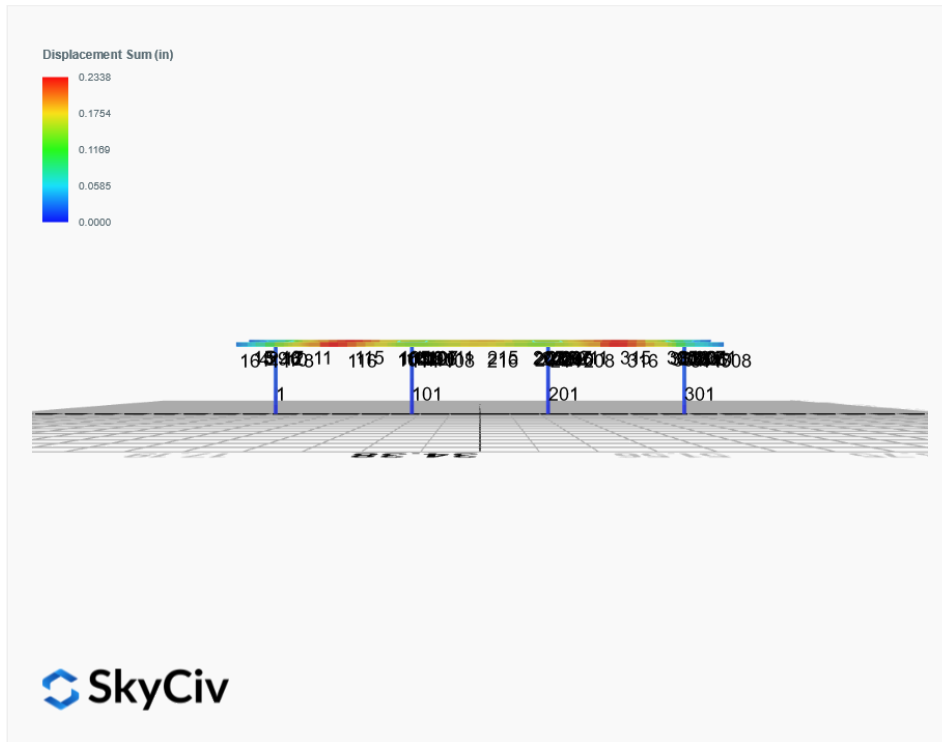


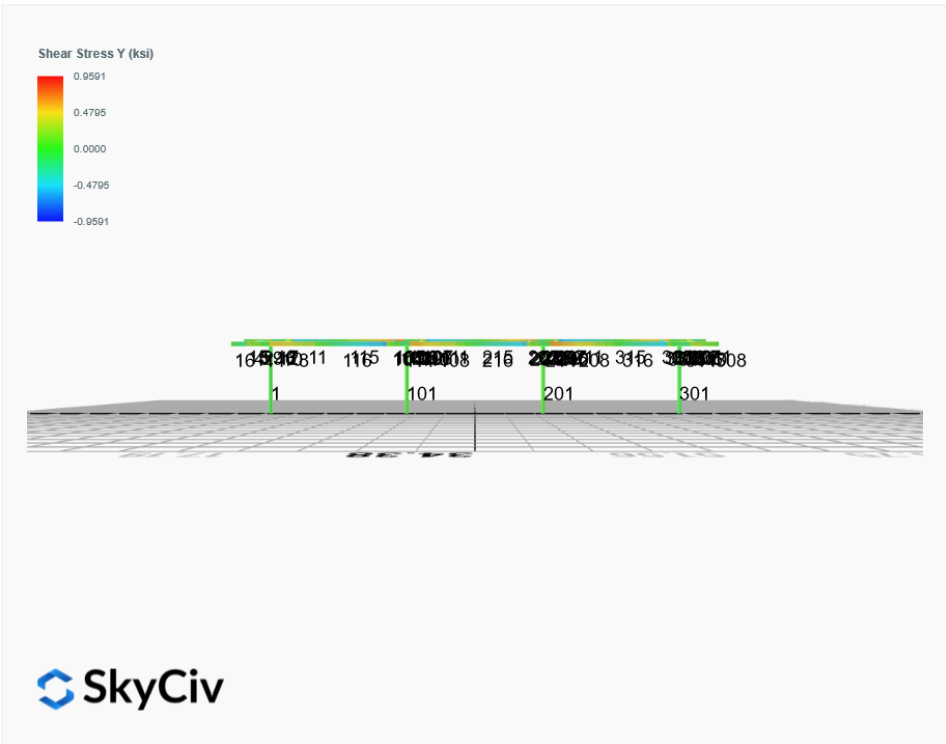
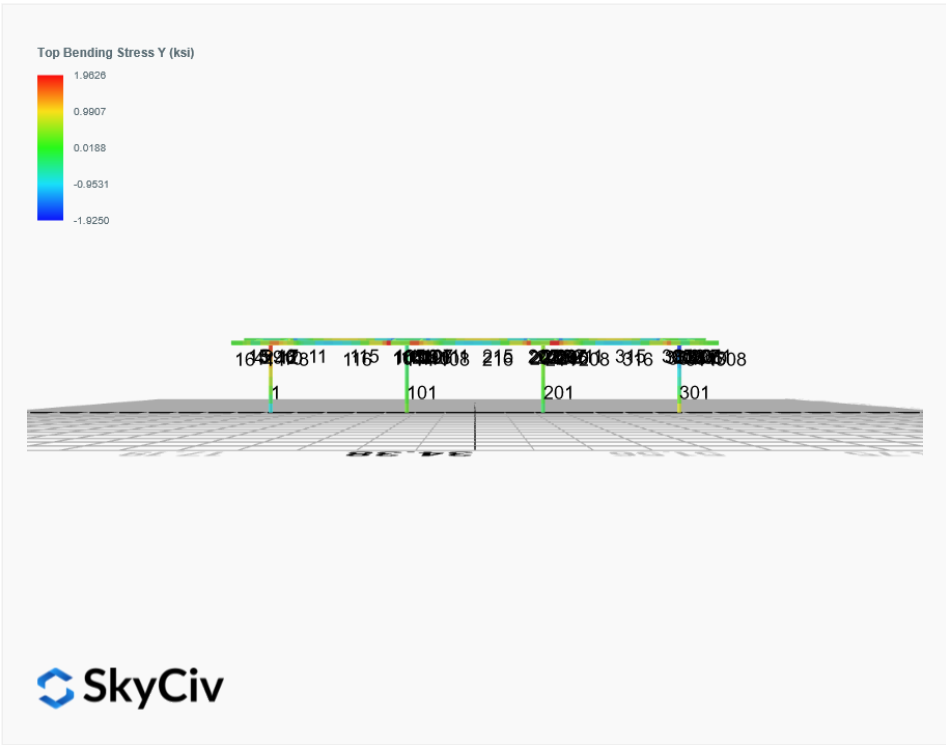
 SkyCiv

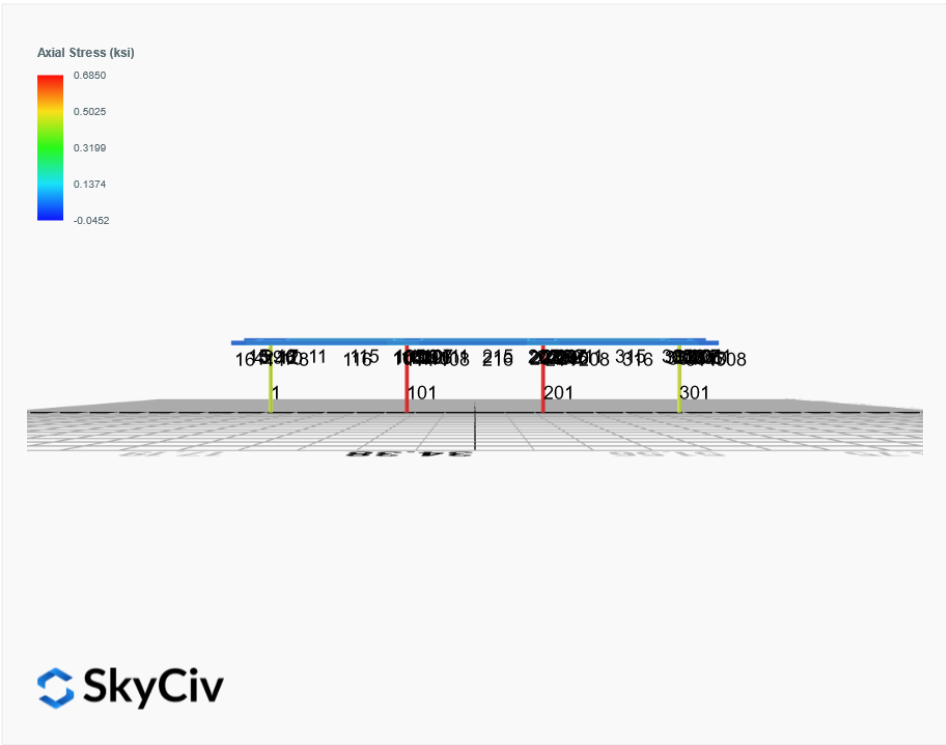


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FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0066	1.7557	0.1005	0.3186	-0.0133	-0.0312
ULS: 2. D + L	0.0066	1.7557	0.1005	0.3186	-0.0133	-0.0312
ULS: 3. D + (S or Lr or R)	0.0190	4.2821	0.2892	0.9172	-0.0382	-0.1426
ULS: 3. D + (S or Lr or R)	0.0066	1.7557	0.1005	0.3186	-0.0133	-0.0312
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0159	3.6505	0.2421	0.7676	-0.0320	-0.1147
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0066	1.7557	0.1005	0.3186	-0.0133	-0.0312
ULS: 5b. D + 0.7E	0.0066	1.7557	0.1005	0.3186	-0.0133	-0.0312
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0159	3.6505	0.2421	0.7676	-0.0320	-0.1147
ULS: 8. 0.6D + 0.7E	0.0040	1.0534	0.0603	0.1912	-0.0080	-0.0187
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2818	4.8505	0.3358	1.0626	-0.0804	3.7221
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2818	4.8505	0.3358	1.0626	-0.0804	3.7221
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0642	0.9235	0.0429	0.1360	-0.0057	2.6830
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2274	-0.2141	-0.0581	-0.1793	0.0487	-9.5672
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2004	5.9716	0.4185	1.3256	-0.0823	2.7002
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2004	5.9716	0.4185	1.3256	-0.0823	2.7002
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0591	3.0263	0.1988	0.6306	-0.0263	1.9209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1815	2.1731	0.1231	0.3942	0.0145	-7.2667
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2097	4.0768	0.2770	0.8766	-0.0636	2.7838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2097	4.0768	0.2770	0.8766	-0.0636	2.7838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0498	1.1316	0.0573	0.1817	-0.0076	2.0044
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1722	0.2783	-0.0185	-0.0548	0.0332	-7.1832
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2845	4.1482	0.2956	0.9352	-0.0751	3.7345
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2845	4.1482	0.2956	0.9352	-0.0751	3.7345
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0616	0.2212	0.0027	0.0086	-0.0004	2.6955
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2248	-0.9164	-0.0983	-0.3067	0.0540	-9.5547

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	8.7275
Shear X	-0.4807
Shear Z	0.6208
Moment X	1.9727
Moment Y (Twist)	0.1413
Moment Z	16.5094

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.9716
Shear X	-0.2845
Shear Z	0.4185
Moment X	1.3256
Moment Y (Twist)	0.0823
Moment Z	9.5672

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	-0.0066	2.4480	-0.0181	-0.0578	0.0040	0.0894
ULS: 2. D + L	-0.0066	2.4480	-0.0181	-0.0578	0.0040	0.0894
ULS: 3. D + (S or Lr or R)	-0.0190	6.2714	-0.0520	-0.1665	0.0115	0.2066
ULS: 3. D + (S or Lr or R)	-0.0066	2.4480	-0.0181	-0.0578	0.0040	0.0894
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0159	5.3156	-0.0435	-0.1393	0.0096	0.1773
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0066	2.4480	-0.0181	-0.0578	0.0040	0.0894
ULS: 5b. D + 0.7E	-0.0066	2.4480	-0.0181	-0.0578	0.0040	0.0894

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0159	5.3156	-0.0435	-0.1393	0.0096	0.1773
ULS: 8. 0.6D + 0.7E	-0.0040	1.4688	-0.0108	-0.0347	0.0024	0.0537
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3987	7.1322	-0.0582	-0.1872	0.0061	5.1006
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3987	7.1322	-0.0582	-0.1872	0.0061	5.1006
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1194	1.1812	-0.0044	-0.0150	-0.0059	3.2856
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2047	-0.5211	0.0025	0.0099	0.0197	-11.6874
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3100	8.8287	-0.0736	-0.2364	0.0112	3.9357
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3100	8.8287	-0.0736	-0.2364	0.0112	3.9357
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0786	4.3655	-0.0333	-0.1072	0.0022	2.5744
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1425	3.0888	-0.0281	-0.0886	0.0214	-8.6553
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3007	5.9611	-0.0481	-0.1548	0.0056	3.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3007	5.9611	-0.0481	-0.1548	0.0056	3.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0879	1.4979	-0.0078	-0.0257	-0.0034	2.4866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1518	0.2212	-0.0026	-0.0070	0.0158	-8.7432
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3961	6.1530	-0.0510	-0.1641	0.0045	5.0648
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3961	6.1530	-0.0510	-0.1641	0.0045	5.0648
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1221	0.2020	0.0028	0.0081	-0.0075	3.2498
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2073	-1.5003	0.0097	0.0330	0.0181	-11.7232

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.9591
Shear X	-0.6661
Shear Z	-0.1099
Moment X	-0.3542
Moment Y (Twist)	0.0367
Moment Z	20.2386

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.8287
Shear X	-0.3987
Shear Z	-0.0736
Moment X	-0.2364
Moment Y (Twist)	0.0214
Moment Z	11.7232

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	-0.0066	2.4480	0.0181	0.0578	-0.0040	0.0894
ULS: 2. D + L	-0.0066	2.4480	0.0181	0.0578	-0.0040	0.0894
ULS: 3. D + (S or Lr or R)	-0.0190	6.2714	0.0520	0.1665	-0.0115	0.2066
ULS: 3. D + (S or Lr or R)	-0.0066	2.4480	0.0181	0.0578	-0.0040	0.0894
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0159	5.3156	0.0435	0.1393	-0.0096	0.1773
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0066	2.4480	0.0181	0.0578	-0.0040	0.0894
ULS: 5b. D + 0.7E	-0.0066	2.4480	0.0181	0.0578	-0.0040	0.0894
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0159	5.3156	0.0435	0.1393	-0.0096	0.1773
ULS: 8. 0.6D + 0.7E	-0.0040	1.4688	0.0108	0.0347	-0.0024	0.0537
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.3987	7.1322	0.0582	0.1872	-0.0061	5.1006
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.3987	7.1322	0.0582	0.1872	-0.0061	5.1006
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.1194	1.1812	0.0044	0.0150	0.0059	3.2856
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2047	-0.5211	-0.0025	-0.0099	-0.0197	-11.6874
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.3100	8.8287	0.0736	0.2364	-0.0112	3.9357
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3100	8.8287	0.0736	0.2364	-0.0112	3.9357
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0786	4.3655	0.0333	0.1072	-0.0022	2.5744
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1425	3.0888	0.0281	0.0886	-0.0214	-8.6553

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	-0.3007	5.9611	0.0481	0.1548	-0.0056	3.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.3007	5.9611	0.0481	0.1548	-0.0056	3.8478
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0879	1.4979	0.0078	0.0257	0.0034	2.4866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1518	0.2212	0.0026	0.0070	-0.0158	-8.7432
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.3961	6.1530	0.0510	0.1641	-0.0045	5.0648
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.3961	6.1530	0.0510	0.1641	-0.0045	5.0648
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.1221	0.2020	-0.0028	-0.0081	0.0075	3.2498
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2073	-1.5003	-0.0097	-0.0330	-0.0181	-11.7232

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	12.9591
Shear X	-0.6661
Shear Z	0.1099
Moment X	0.3542
Moment Y (Twist)	0.0367
Moment Z	20.2386

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.8287
Shear X	-0.3987
Shear Z	0.0736
Moment X	0.2364
Moment Y (Twist)	0.0214
Moment Z	11.7232

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	0.0066	1.7557	-0.1005	-0.3186	0.0133	-0.0312
ULS: 2. D + L	0.0066	1.7557	-0.1005	-0.3186	0.0133	-0.0312
ULS: 3. D + (S or Lr or R)	0.0190	4.2821	-0.2892	-0.9172	0.0382	-0.1426
ULS: 3. D + (S or Lr or R)	0.0066	1.7557	-0.1005	-0.3186	0.0133	-0.0312
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0159	3.6505	-0.2421	-0.7676	0.0320	-0.1147
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0066	1.7557	-0.1005	-0.3186	0.0133	-0.0312
ULS: 5b. D + 0.7E	0.0066	1.7557	-0.1005	-0.3186	0.0133	-0.0312
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0159	3.6505	-0.2421	-0.7676	0.0320	-0.1147
ULS: 8. 0.6D + 0.7E	0.0040	1.0534	-0.0603	-0.1912	0.0080	-0.0187
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.2818	4.8505	-0.3358	-1.0626	0.0804	3.7221
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.2818	4.8505	-0.3358	-1.0626	0.0804	3.7221
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.0642	0.9235	-0.0429	-0.1360	0.0057	2.6830
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.2274	-0.2141	0.0581	0.1793	-0.0487	-9.5672
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2004	5.9716	-0.4185	-1.3256	0.0823	2.7002
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2004	5.9716	-0.4185	-1.3256	0.0823	2.7002
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0591	3.0263	-0.1988	-0.6306	0.0263	1.9209
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1815	2.1731	-0.1231	-0.3942	-0.0145	-7.2667
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.2097	4.0768	-0.2770	-0.8766	0.0636	2.7838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.2097	4.0768	-0.2770	-0.8766	0.0636	2.7838
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.0498	1.1316	-0.0573	-0.1817	0.0076	2.0044
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.1722	0.2783	0.0185	0.0548	-0.0332	-7.1832
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.2845	4.1482	-0.2956	-0.9352	0.0751	3.7345
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.2845	4.1482	-0.2956	-0.9352	0.0751	3.7345
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.0616	0.2212	-0.0027	-0.0086	0.0004	2.6955
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.2248	-0.9164	0.0983	0.3067	-0.0540	-9.5547

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	8.7275
Shear X	-0.4807
Shear Z	-0.6208
Moment X	-1.9727
Moment Y (Twist)	0.1414
Moment Z	16.5095

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

Result	Value (kip, kip-ft)
Axial	5.9716
Shear X	-0.2845
Shear Z	-0.4185
Moment X	-1.3256
Moment Y (Twist)	0.0823
Moment Z	9.5672

Project Details

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

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

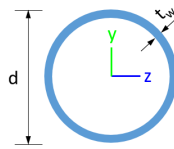


Design Input Information

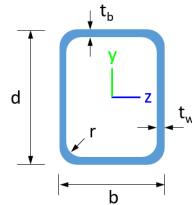
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

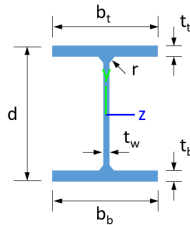
Section Dimensions



ID	Name	d (in)	t_w (in)					
2	2in Pipe Sch 80	2.38	0.22					
5	4in Pipe Sch 80	4.50	0.34					
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
7	6in Pipe Sch 40	5.58	56.28	28.14	28.14	0.00	11.28	11.28

108	19	1.33	1.33	2.0 5	2.36,2.36,2.36,2.35,2.36,2.36,2.32,2.32,1.06,1.62,2.32,2.32,1.68,1.35,2.34,2.34,2.30,2.10,2.3 3,2.33,1.29,1.69,2.32,2.32,1.89,1.25	3 0 0	2 0 0	1
109	2	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
110	16	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.88,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.67,1.67,1.6 7,1.67,1.77,1.68,1.67,1.67,1.65,1.68	3 0 0	2 0 0	1
111	19	1.33	1.33	2.0 5	2.37,2.37,2.37,2.37,2.37,2.37,2.26,2.26,2.07,1.67,2.23,2.23,1.76,1.88,2.38,2.38,2.16,1.86,2.2 8,2.28,2.07,1.61,2.21,2.21,1.59,1.95	3 0 0	2 0 0	1
112	5	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
113	19	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.22,1.06,1.06,1.17,1.13,1.06,1.06,1.06,1.04,1.0 6,1.06,1.07,1.27,1.06,1.06,1.30,1.11	3 0 0	2 0 0	1
114	19	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.01,1.29,1.05,1.05,1.31,3.64,1.05,1.05,1.05,1.06,1.0 5,1.05,1.02,1.21,1.05,1.05,1.17,1.73	3 0 0	2 0 0	1
115	19	6.63	6.63	10. 20	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.13,1.10,1.15,1.15,1.11,1.12,1.16,1.16,1.15,1.25,1.1 6,1.16,1.13,1.09,1.15,1.15,1.09,1.12	3 0 0	2 0 0	1
116	19	6.63	6.63	10. 20	1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.18,2.44,1.09,1.18,1.18,1.10,1.08,1.17,1.17,1.19,1.14,1.1 8,1.18,1.64,1.10,1.18,1.18,1.12,1.08	3 0 0	2 0 0	1
201	7	21.5 3	21.5 3	10. 25	-	3 0 0	2 0 0	1
202	5	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
203	16	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.18,1.18,1.18,1.1 8,1.18,1.19,1.17,1.18,1.18,1.19,1.18	3 0 0	2 0 0	1
204	16	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.88,1.68,1.67,1.67,1.64,1.68,1.67,1.67,1.67,1.67,1.6 7,1.67,1.77,1.68,1.67,1.67,1.65,1.68	3 0 0	2 0 0	1
205	16	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.67,1.6 7,1.67,1.68,1.66,1.67,1.67,1.68,1.66	3 0 0	2 0 0	1
206	16	0.92	0.92	1.4 2	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.1 8,1.18,1.19,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1
207	16	1.52	1.52	2.3 3	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.67,1.67,1.67,1.6 7,1.67,1.68,1.66,1.67,1.67,1.68,1.66	3 0 0	2 0 0	1
208	19	1.33	1.33	2.0 5	2.08,2.08,2.08,2.08,2.08,2.08,2.09,2.09,1.21,1.38,2.09,2.09,1.42,1.20,2.08,2.08,2.10,1.81,2.0 9,2.09,1.55,1.42,2.09,2.09,1.51,1.13	3 0 0	2 0 0	1
209	2	2.60	2.60	4.0 0	-	3 0 0	2 0 0	1
210	16	2.44	2.44	3.7 5	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.67,1.77,1.68,1.67,1.67,1.64,1.69,1.67,1.67,1.67,1.67,1.6 7,1.67,1.74,1.68,1.67,1.67,1.65,1.69	3 0 0	2 0 0	1
211	19	1.33	1.33	2.0 5	2.07,2.07,2.07,2.07,2.07,2.07,1.99,1.99,1.64,1.42,1.97,1.97,1.46,1.52,2.05,2.05,1.94,2.30,2.0 0,2.00,1.67,1.38,1.96,1.96,1.37,1.54	3 0 0	2 0 0	1
212	5	1.30	1.30	2.0 0	-	3 0 0	2 0 0	1
213	19	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.06,1.06,1.07,1.22,1.06,1.06,1.17,1.13,1.06,1.06,1.06,1.04,1.0 6,1.06,1.07,1.27,1.06,1.06,1.30,1.11	3 0 0	2 0 0	1
214	19	4.88	4.00	7.5 0	1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.05,1.01,1.29,1.05,1.05,1.31,3.64,1.05,1.05,1.05,1.06,1.0 5,1.05,1.02,1.21,1.05,1.05,1.17,1.73	3 0 0	2 0 0	1
215	19	6.63	6.63	10. 20	1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.15,1.12,1.17,1.17,1.13,1.14,1.18,1.18,1.17,1.38,1.1 7,1.17,1.15,1.12,1.17,1.17,1.12,1.14	3 0 0	2 0 0	1
216	19	6.63	6.63	10. 20	1.18,1.18,1.18,1.18,1.18,1.18,1.19,1.19,1.18,1.12,1.19,1.19,1.12,1.09,1.19,1.19,1.20,1.16,1.1 9,1.19,2.81,1.12,1.19,1.19,1.14,1.08	3 0 0	2 0 0	1
301	7	21.5 3	21.5 3	10. 25	-	3 0 0	2 0 0	1

103	110.10	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	23.78	6.12	40.24	43.62
114	133.20	104.94	23.04	6.12	40.24	43.62
115	133.20	69.16	16.91	6.12	40.24	43.62
116	133.20	69.16	16.65	6.12	40.24	43.62
201	251.16	95.26	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	23.78	6.12	40.24	43.62
214	133.20	104.94	23.04	6.12	40.24	43.62
215	133.20	69.16	17.30	6.12	40.24	43.62
216	133.20	69.16	16.68	6.12	40.24	43.62
301	251.16	95.26	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	121.82	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	121.82	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	31.66	6.12	40.24	43.62
314	133.20	104.94	32.35	6.12	40.24	43.62
315	133.20	69.16	16.68	6.12	40.24	43.62
316	133.20	69.16	16.69	6.12	40.24	43.62

Design Ratio

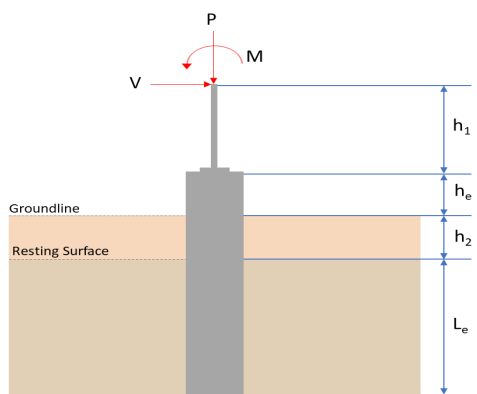
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.092	0.390	0.104	0.006	0.008	0.398	#32	0.575	Not Required	Pass
2	0.002	0.233	0.018	0.057	0.003	0.246	#13	0.053	Not Required	Pass
3	0.002	0.405	0.022	0.040	0.006	0.428	#21	0.045	Not Required	Pass
4	0.001	0.201	0.027	0.040	0.008	0.430	#21	0.120	Not Required	Pass

4	U.001	U.391	U.037	U.040	U.008	U.428	#21	U.120	Not Required	Pass
5	0.001	0.251	0.014	0.041	0.006	0.265	#21	0.074	Not Required	Pass
6	0.001	0.603	0.040	0.062	0.010	0.643	#21	0.045	Not Required	Pass
7	0.002	0.373	0.044	0.060	0.012	0.391	#21	0.074	Not Required	Pass
8	0.003	0.150	0.052	0.038	0.003	0.170	#21	0.095	Not Required	Pass
9	0.002	0.079	0.033	0.004	0.003	0.112	#21	0.204	Not Required	Pass
10	0.002	0.576	0.037	0.058	0.008	0.597	#21	0.080	Not Required	Pass
11	0.003	0.150	0.052	0.040	0.003	0.165	#21	0.095	Not Required	Pass
12	0.002	0.432	0.024	0.086	0.005	0.450	#21	0.053	Not Required	Pass
13	0.004	0.070	0.086	0.053	0.004	0.120	#16	0.286	Not Required	Pass
14	0.003	0.075	0.082	0.051	0.004	0.115	#16	0.190	Not Required	Pass
15	0.000	0.005	0.001	0.008	0.000	0.006	#21	Not Required	Not Required	Pass
16	0.000	0.005	0.001	0.007	0.000	0.006	#21	Not Required	Not Required	Pass
101	0.136	0.478	0.018	0.009	0.001	0.479	#16	0.575	Not Required	Pass
102	0.000	0.531	0.033	0.110	0.005	0.556	#21	0.053	Not Required	Pass
103	0.002	0.770	0.015	0.078	0.003	0.786	#21	0.045	Not Required	Pass
104	0.002	0.754	0.028	0.076	0.006	0.768	#21	0.120	Not Required	Pass
105	0.002	0.477	0.034	0.077	0.009	0.488	#21	0.074	Not Required	Pass
106	0.002	0.739	0.012	0.074	0.005	0.743	#21	0.045	Not Required	Pass
107	0.002	0.459	0.024	0.074	0.006	0.464	#21	0.074	Not Required	Pass
108	0.002	0.057	0.023	0.045	0.003	0.081	#21	0.095	Not Required	Pass
109	0.003	0.086	0.014	0.001	0.001	0.101	#21	0.204	Not Required	Pass
110	0.002	0.717	0.029	0.072	0.007	0.740	#21	0.080	Not Required	Pass
111	0.003	0.053	0.022	0.046	0.003	0.077	#21	0.095	Not Required	Pass
112	0.001	0.496	0.032	0.104	0.006	0.520	#21	0.053	Not Required	Pass
113	0.004	0.249	0.072	0.067	0.003	0.291	#21	0.286	Not Required	Pass
114	0.004	0.256	0.075	0.065	0.004	0.299	#21	0.286	Not Required	Pass
115	0.005	0.358	0.035	0.053	0.003	0.393	#21	0.473	Not Required	Pass
116	0.005	0.348	0.036	0.053	0.003	0.383	#21	0.473	Not Required	Pass
201	0.136	0.478	0.018	0.009	0.001	0.479	#16	0.575	Not Required	Pass
202	0.001	0.496	0.032	0.104	0.006	0.520	#21	0.053	Not Required	Pass
203	0.002	0.739	0.012	0.074	0.005	0.743	#21	0.045	Not Required	Pass
204	0.002	0.717	0.029	0.072	0.007	0.740	#21	0.080	Not Required	Pass
205	0.002	0.459	0.024	0.074	0.006	0.464	#21	0.074	Not Required	Pass
206	0.002	0.770	0.015	0.078	0.003	0.786	#21	0.045	Not Required	Pass
207	0.002	0.477	0.034	0.077	0.009	0.488	#21	0.074	Not Required	Pass
208	0.003	0.076	0.036	0.053	0.003	0.082	#24	0.095	Not Required	Pass
209	0.003	0.086	0.014	0.001	0.001	0.101	#21	0.204	Not Required	Pass
210	0.002	0.754	0.028	0.076	0.006	0.768	#21	0.120	Not Required	Pass
211	0.003	0.084	0.035	0.053	0.003	0.096	#32	0.095	Not Required	Pass
212	0.000	0.531	0.033	0.110	0.005	0.556	#21	0.053	Not Required	Pass
213	0.004	0.249	0.072	0.067	0.003	0.291	#21	0.286	Not Required	Pass
214	0.004	0.256	0.075	0.065	0.004	0.299	#21	0.286	Not Required	Pass
215	0.005	0.220	0.033	0.046	0.003	0.253	#21	0.473	Not Required	Pass
216	0.004	0.201	0.033	0.045	0.003	0.234	#21	0.473	Not Required	Pass
301	0.092	0.390	0.104	0.006	0.008	0.398	#32	0.575	Not Required	Pass
302	0.002	0.432	0.024	0.086	0.005	0.450	#21	0.053	Not Required	Pass
303	0.001	0.603	0.040	0.062	0.010	0.643	#21	0.045	Not Required	Pass
304	0.002	0.576	0.037	0.058	0.008	0.597	#21	0.080	Not Required	Pass
305	0.002	0.373	0.044	0.060	0.012	0.391	#21	0.074	Not Required	Pass
306	0.002	0.405	0.022	0.040	0.006	0.428	#21	0.045	Not Required	Pass
307	0.001	0.251	0.014	0.041	0.006	0.265	#21	0.074	Not Required	Pass
308	0.000	0.005	0.001	0.007	0.000	0.006	#21	Not Required	Not Required	Pass
309	0.002	0.079	0.033	0.004	0.003	0.112	#21	0.204	Not Required	Pass

310	0.001	0.391	0.037	0.040	0.008	0.428	#21	0.120	Not Required	Pass
311	0.000	0.005	0.001	0.008	0.000	0.006	#21	Not Required	Not Required	Pass
312	0.002	0.233	0.018	0.057	0.003	0.246	#13	0.053	Not Required	Pass
313	0.004	0.070	0.086	0.053	0.004	0.121	#16	0.190	Not Required	Pass
314	0.003	0.075	0.082	0.051	0.004	0.115	#16	0.286	Not Required	Pass
315	0.005	0.389	0.052	0.040	0.003	0.423	#21	0.473	Not Required	Pass
316	0.005	0.379	0.052	0.038	0.003	0.416	#21	0.473	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ 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="414 1097 1189 1198"> <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="670 1288 933 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.972</td> <td>8.727</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.284</td> <td>-0.481</td> </tr> <tr> <td>V_z (kip)</td> <td>0.419</td> <td>0.621</td> </tr> <tr> <td>M_x (kipft)</td> <td>1.326</td> <td>1.973</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.567</td> <td>16.509</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.972	8.727	V_x (kip)	-0.284	-0.481	V_z (kip)	0.419	0.621	M_x (kipft)	1.326	1.973	M_z (kipft)	9.567	16.509	
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M_z (kipft)	9.567	16.509																										
	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.284 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.045223 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(9.567 \text{ kipft}) + ((-0.284 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5234 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7756 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.419 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.06672 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.326 \text{ kipft}) + ((0.419 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21115 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 3.0802 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.7756 \text{ ft}), (3.0802 \text{ ft})]$$

$$L_{e,req} = 4.776 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.776 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9552$$

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

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

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

$$q = \frac{(5.972 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37325 \text{ kip/ft}^2$$

$$q = 0.37325 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37325 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18663$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.045223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.5234 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5234 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.5234 \text{ kipft/ft})) + (4 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3709 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5234 \text{ kipft/ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.5234 \text{ kipft/ft})) + (2 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.21364 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.5234 \text{ kipft/ft})) + ((-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.67697 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3709 \text{ ft})}{2}$$

$$p_a = 0.25281 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21364 \text{ kip/ft}^2)}{(0.25281 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84504$$

p_a - Allowable lateral soil pressure at depth L_e ,

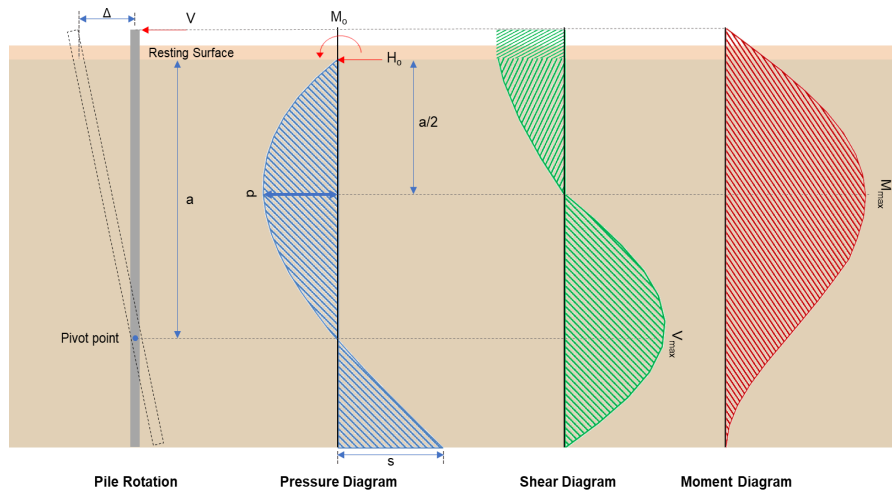
Status: **PASS**
Ratio: **0.850**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.67697 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90262$	<p>Status: PASS Ratio: 0.900</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.06672 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.21115 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.21115 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.06672 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.21115 \text{ kipft/ft})) + (4 \times (0.06672 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5471 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.21115 \text{ kipft/ft})) + (3 \times (0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.21115 \text{ kipft/ft})) + (2 \times (0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.078548 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.21115 \text{ kipft/ft})) + ((0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.18141 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5471 \text{ ft})}{2}$ $p_a = 0.26603 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.078548 \text{ kip/ft}^2)}{(0.26603 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.29526$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.300</p>

$$Ratio = \frac{(0.18141 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.24189$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.481 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.076592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.509 \text{ kipft}) + ((-0.481 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6288 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.6288 \text{ kipft/ft})}{(-0.076592 \text{ kip/ft})}$$

$$E = 34.322 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6288 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.076592 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.6288 \text{ kipft/ft})) + (4 \times (-0.076592 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3702 \text{ ft}$$

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

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

$$V_{max} = ((-0.076592 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3702 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3702 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.9332 \text{ kip}$$

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

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

$$M_{max} = ((-0.076592 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(34.322 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 9.692 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.621 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.098885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.973 \text{ kipft}) + ((0.621 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.31417 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.31417 \text{ kipft/ft})}{(0.098885 \text{ kip/ft})}$$

$$E = 3.1771 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.31417 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.098885 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.31417 \text{ kipft/ft})) + (4 \times (0.098885 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5467 \text{ ft}$$

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

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

$$V_{max} = ((0.098885 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5467 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5467 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.70736 \text{ kip}$$

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

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

$$M_{max} = ((0.098885 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(3.1771 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5914 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.727 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.306 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.306 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(8.727 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0032622$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.727 \text{ kip} \rightarrow 8727 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8727 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.65 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

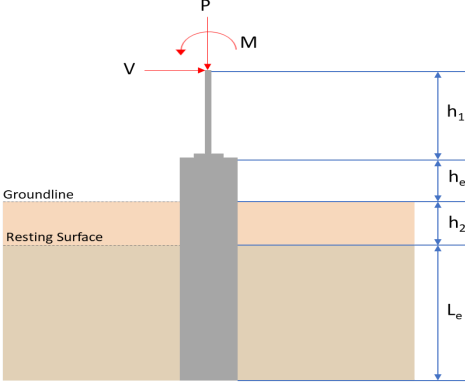
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.65 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.65 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.65 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.85 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.9332 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.9332 \text{ kip})}{(110.85 \text{ kip})}$ $\text{Ratio} = 0.035481$ <p>Considering z-direction:</p> <p>$V_{max} = 0.70736 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.70736 \text{ kip})}{(110.85 \text{ kip})}$ $\text{Ratio} = 0.0063811$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.010</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 9.692 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(9.692 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.03883$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 1.5914 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5914 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0063758$	<p>Status: PASS Ratio: 0.010</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</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ 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>5.972</td> <td>8.727</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.284</td> <td>-0.481</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.419</td> <td>-0.621</td> </tr> <tr> <td>M_x (kipft)</td> <td>-1.326</td> <td>-1.973</td> </tr> <tr> <td>M_z (kipft)</td> <td>9.567</td> <td>16.509</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	5.972	8.727	V_x (kip)	-0.284	-0.481	V_z (kip)	-0.419	-0.621	M_x (kipft)	-1.326	-1.973	M_z (kipft)	9.567	16.509	
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	<p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.284 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.045223 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(9.567 \text{ kipft}) + ((-0.284 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.5234 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.7756 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.419 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.06672 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.326 \text{ kipft}) + ((-0.419 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21115 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0542 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.7756 \text{ ft}), (2.0542 \text{ ft})]$$

$$L_{e,req} = 4.776 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.776 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9552$$

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

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

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

$$q = \frac{(5.972 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.37325 \text{ kip/ft}^2$$

$$q = 0.37325 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.37325 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18663$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.045223 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.5234 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.5234 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.5234 \text{ kipft/ft})) + (4 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3709 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.5234 \text{ kipft/ft})) + (3 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.5234 \text{ kipft/ft})) + (2 \times (-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.21364 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.5234 \text{ kipft/ft})) + ((-0.045223 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.67697 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3709 \text{ ft})}{2}$$

$$p_a = 0.25281 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21364 \text{ kip/ft}^2)}{(0.25281 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84504$$

p_a - Allowable lateral soil pressure at depth L_e ,

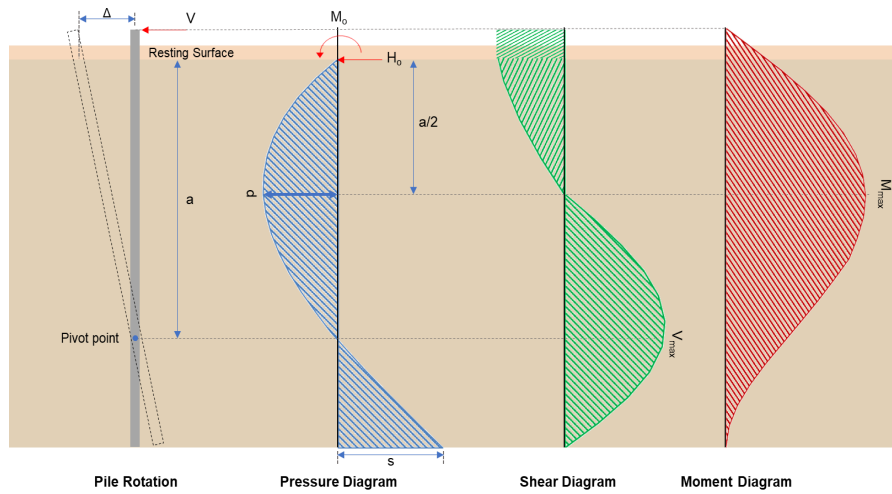
Status: **PASS**
Ratio: **0.850**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.67697 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.90262$	Status: PASS Ratio: 0.900
	<p>Considering z-direction:</p> <p>$H_o = -0.06672 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.21115 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.21115 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.06672 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.21115 \text{ kipft/ft})) + (4 \times (-0.06672 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5471 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.21115 \text{ kipft/ft})) + (3 \times (-0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.21115 \text{ kipft/ft})) + (2 \times (-0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = -0.021685 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.21115 \text{ kipft/ft})) + ((-0.06672 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.021287 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5471 \text{ ft})}{2}$ $p_a = 0.26603 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.021685 \text{ kip/ft}^2)}{(0.26603 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.081514$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.080

$$Ratio = \frac{(0.021287 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.028382$$

Status: **PASS**
Ratio: **0.030**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.481 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.076592 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.509 \text{ kipft}) + ((-0.481 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6288 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(2.6288 \text{ kipft/ft})}{(-0.076592 \text{ kip/ft})}$$

$$E = 34.322 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6288 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.076592 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (2.6288 \text{ kipft/ft})) + (4 \times (-0.076592 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3702 \text{ ft}$$

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

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

$$V_{max} = ((-0.076592 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3702 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3702 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 3.9332 \text{ kip}$$

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

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

$$M_{max} = ((-0.076592 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(34.322 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (34.322 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3702 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 9.692 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.621 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.098885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.973 \text{ kipft}) + ((-0.621 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.31417 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.31417 \text{ kipft/ft})}{(-0.098885 \text{ kip/ft})}$$

$$E = 3.1771 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.31417 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.098885 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.31417 \text{ kipft/ft})) + (4 \times (-0.098885 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.5467 \text{ ft}$$

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

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

$$V_{max} = ((-0.098885 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5467 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5467 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.70736 \text{ kip}$$

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

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

$$M_{max} = ((-0.098885 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(3.1771 \text{ ft})}{(5 \text{ ft})} + \frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.1771 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.5467 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.5914 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.727 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.306 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.306 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(8.727 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0032622$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.727 \text{ kip} \rightarrow 8727 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8727 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.65 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

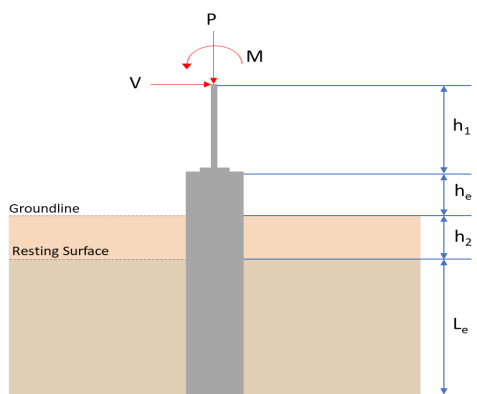
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.65 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.65 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.65 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.85 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 3.9332 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(3.9332 \text{ kip})}{(110.85 \text{ kip})}$ $\text{Ratio} = 0.035481$ <p>Considering z-direction:</p> <p>$V_{max} = 0.70736 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.70736 \text{ kip})}{(110.85 \text{ kip})}$ $\text{Ratio} = 0.0063811$	<p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.010</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 9.692 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(9.692 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.03883$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 1.5914 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.5914 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0063758$	<p>Status: PASS Ratio: 0.010</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="414 1097 1189 1198"> <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="670 1288 933 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.829</td> <td>12.959</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.399</td> <td>-0.666</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.074</td> <td>-0.110</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.236</td> <td>-0.354</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.723</td> <td>20.239</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)	8.829	12.959	V_x (kip)	-0.399	-0.666	V_z (kip)	-0.074	-0.110	M_x (kipft)	-0.236	-0.354	M_z (kipft)	11.723	20.239	
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M_z (kipft)	11.723	20.239																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.399 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.063535 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.723 \text{ kipft}) + ((-0.399 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.8667 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.0661 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.074 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.011783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.236 \text{ kipft}) + ((-0.074 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

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:

$L_{e,z} = 1.2808 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0661 \text{ ft}), (1.2808 \text{ ft})]$$

$$L_{e,req} = 5.066 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.066 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.96495$$

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

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

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

$$q = \frac{(8.829 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55181 \text{ kip/ft}^2$$

$$q = 0.00101 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55181 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27591$$

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:

$H_o = -0.063535 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.8667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8667 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.8667 \text{ kipft/ft})) + (4 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5466 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.8667 \text{ kipft/ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.8667 \text{ kipft/ft})) + (2 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.23064 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.8667 \text{ kipft/ft})) + ((-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74011 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5466 \text{ ft})}{2}$$

$$p_a = 0.26599 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23064 \text{ kip/ft}^2)}{(0.26599 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86708$$

p_a - Allowable lateral soil pressure at depth L_e ,

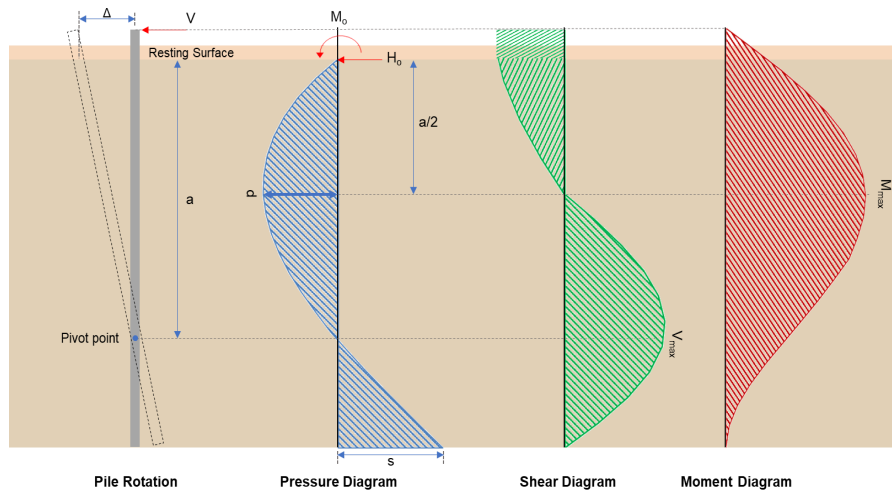
Status: **PASS**
Ratio: **0.870**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.25 \text{ ft})$ $p_s = 0.7875 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74011 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93982$	Status: PASS Ratio: 0.940
	<p>Considering z-direction:</p> <p>$H_o = -0.011783 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.03758 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.03758 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.011783 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.03758 \text{ kipft/ft})) + (4 \times (-0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))}$ $a = 3.7289 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.03758 \text{ kipft/ft})) + (3 \times (-0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.03758 \text{ kipft/ft})) + (2 \times (-0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$ $p = -0.0030809 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.03758 \text{ kipft/ft})) + ((-0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$ $s = 0.0028944 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.7289 \text{ ft})}{2}$ $p_a = 0.27967 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0030809 \text{ kip/ft}^2)}{(0.27967 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.011016$ <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

$$Ratio = \frac{(0.0028944 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$Ratio = 0.0036754$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.666 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.239 \text{ kipft}) + ((-0.666 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2228 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.2228 \text{ kipft/ft})}{(-0.10605 \text{ kip/ft})}$$

$$E = 30.389 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2228 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.10605 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.2228 \text{ kipft/ft})) + (4 \times (-0.10605 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5452 \text{ ft}$$

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

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

$$V_{max} = ((-0.10605 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.6348 \text{ kip}$$

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

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

$$M_{max} = ((-0.10605 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(30.389 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.962 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.11 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.017516 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.354 \text{ kipft}) + ((-0.11 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.056369 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.056369 \text{ kipft/ft})}{(-0.017516 \text{ kip/ft})}$$

$$E = 3.2182 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.056369 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.017516 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.056369 \text{ kipft/ft})) + (4 \times (-0.017516 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7279 \text{ ft}$$

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

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

$$V_{max} = ((-0.017516 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7279 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7279 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.12254 \text{ kip}$$

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

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

$$M_{max} = ((-0.017516 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(3.2182 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.28876 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.959 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.166 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.166 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(12.959 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0048442$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.959 \text{ kip} \rightarrow 12959 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12959 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.21 \text{ kip}$$

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

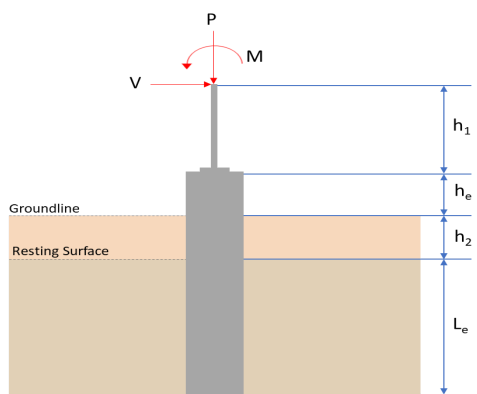
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.21 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.21 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.21 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.22 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.6348 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.6348 \text{ kip})}{(111.22 \text{ kip})}$ $\text{Ratio} = 0.041672$ <p>Considering z-direction:</p> <p>$V_{max} = 0.12254 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.12254 \text{ kip})}{(111.22 \text{ kip})}$ $\text{Ratio} = 0.0011018$	<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: $M_{max} = 11.962 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.962 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.047923$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.28876 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.28876 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0011569$	<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="414 1097 1189 1198"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="670 1288 933 1456"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.829</td> <td>12.959</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.399</td> <td>-0.666</td> </tr> <tr> <td>V_z (kip)</td> <td>0.074</td> <td>0.110</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.236</td> <td>0.354</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.723</td> <td>20.239</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_n) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	8.829	12.959	V_x (kip)	-0.399	-0.666	V_z (kip)	0.074	0.110	M_x (kipft)	0.236	0.354	M_z (kipft)	11.723	20.239	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.399 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.063535 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(11.723 \text{ kipft}) + ((-0.399 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.8667 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.0661 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.074 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.011783 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.236 \text{ kipft}) + ((0.074 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

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:

$L_{e,z} = 1.6059 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.0661 \text{ ft}), (1.6059 \text{ ft})]$$

$$L_{e,req} = 5.066 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.066 \text{ ft})}{(5.25 \text{ ft})}$$

$$\text{Ratio} = 0.96495$$

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

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

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

$$q = \frac{(8.829 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.55181 \text{ kip/ft}^2$$

$$q = 0.00101 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.55181 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.27591$$

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:

$H_o = -0.063535 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.8667 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.8667 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (1.8667 \text{ kipft/ft})) + (4 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5466 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (1.8667 \text{ kipft/ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (1.8667 \text{ kipft/ft})) + (2 \times (-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.23064 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (1.8667 \text{ kipft/ft})) + ((-0.063535 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.74011 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.5466 \text{ ft})}{2}$$

$$p_a = 0.26599 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23064 \text{ kip/ft}^2)}{(0.26599 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86708$$

p_a - Allowable lateral soil pressure at depth L_e ,

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

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.74011 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93982$$

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

Considering z-direction:

$H_o = 0.011783 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.03758 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.03758 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.011783 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.03758 \text{ kipft/ft})) + (4 \times (0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7289 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.03758 \text{ kipft/ft})) + (3 \times (0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]^2}{(5.25 \text{ ft})^2 [(3 \times (0.03758 \text{ kipft/ft})) + (2 \times (0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]}$$

$$p = 0.012984 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.03758 \text{ kipft/ft})) + ((0.011783 \text{ kip/ft}) \times (5.25 \text{ ft}))]}{(5.25 \text{ ft})^2}$$

$$s = 0.029828 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.7289 \text{ ft})}{2}$$

$$p_a = 0.27967 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.012984 \text{ kip/ft}^2)}{(0.27967 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.046427$$

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

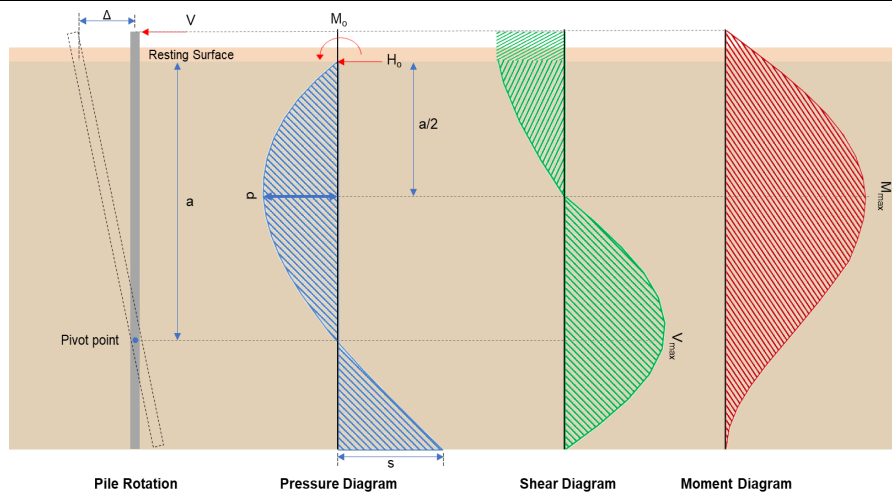
$$\text{Ratio} = \frac{s}{p_s}$$

Status: **PASS**
Ratio: **0.050**

$$\text{Ratio} = \frac{(0.029828 \text{ kip/ft}^2)}{(0.7875 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.037877$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.666 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.239 \text{ kipft}) + ((-0.666 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2228 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.2228 \text{ kipft/ft})}{(-0.10605 \text{ kip/ft})}$$

$$E = 30.389 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2228 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (-0.10605 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (3.2228 \text{ kipft/ft})) + (4 \times (-0.10605 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.5452 \text{ ft}$$

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

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

$$V_{max} = ((-0.10605 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.6348 \text{ kip}$$

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

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

$$M_{max} = ((-0.10605 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(30.389 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (30.389 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.5452 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.962 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.11 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.017516 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.354 \text{ kipft}) + ((0.11 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.056369 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.056369 \text{ kipft/ft})}{(0.017516 \text{ kip/ft})}$$

$$E = 3.2182 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.056369 \text{ kipft/ft}) \times (5.25 \text{ ft})) + (3 \times (0.017516 \text{ kip/ft}) \times (5.25 \text{ ft})^2)}{(6 \times (0.056369 \text{ kipft/ft})) + (4 \times (0.017516 \text{ kip/ft}) \times (5.25 \text{ ft}))}$$

$$a = 3.7279 \text{ ft}$$

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

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

$$V_{max} = ((0.017516 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7279 \text{ ft})}{(5.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7279 \text{ ft})}{(5.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.12254 \text{ kip}$$

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

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

$$M_{max} = ((0.017516 \text{ kip/ft}) \times (48 \text{ in}) \times (5.25 \text{ ft})) \times \left[\left(\frac{(3.2182 \text{ ft})}{(5.25 \text{ ft})} + \frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 3 \right) \times \left(\frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.2182 \text{ ft})}{(5.25 \text{ ft})} + 2 \right) \times \left(\frac{(3.7279 \text{ ft})}{2 \times (5.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.28876 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.959 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.166 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.166 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(12.959 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0048442$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.959 \text{ kip} \rightarrow 12959 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12959 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.21 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.21 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.21 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.21 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.22 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.6348 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.6348 \text{ kip})}{(111.22 \text{ kip})}$ $\text{Ratio} = 0.041672$ <p>Considering z-direction:</p> <p>$V_{max} = 0.12254 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.12254 \text{ kip})}{(111.22 \text{ kip})}$ $\text{Ratio} = 0.0011018$	<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: $M_{max} = 11.962 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.962 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.047923$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.28876 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.28876 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0011569$	<p>Status: PASS Ratio: 0.000</p>