

Your Project Calculations



Project Name: Off Grid - Heath Texas-CU-A

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Off%20Grid%20-%20Heath%20Texas-CU-A&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/1_2024

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=ZubNafqeNvM6EYGIU3mLmyVNTG6vujNSpBR4vAFttTegLf2E0zl6TA4toKB334XG

Array Specification

Product:	Beam
Unique ID:	4P-19.75-6TOP-HD-45-L-4Hx12W-F0L0
Duty Classification:	HD
Module Width:	41.50 in
Module Length:	75.00in
Number of Rows:	4
Number of Columns:	12
Total Number of Modules:	48
Desired Tilt Angle:	40
Front Edge Clearance:	3
Total Array Height at Tilt:	11.95 ft
Total Frame Length:	74.25 ft
Frame Weight:	3201 lbs
Array Dimensions N/S:	14.00 ft
Array Dimensions E/W:	76.00 ft
Rail Length:	168.00 in
Rail Spacing:	3.17 ft
Rail Check:	PASS (42% utilized)

Support Specifications

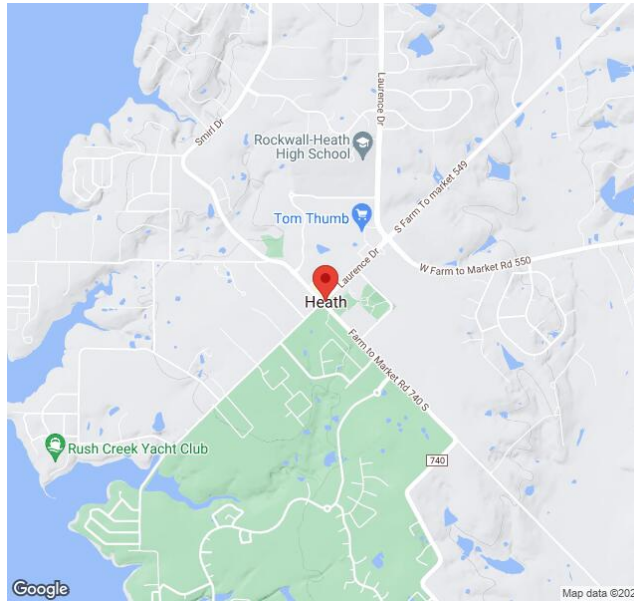
Pole Size:	6in Pipe Sch 80
Pole Length above Grade:	7.50 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.00 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.00 ft
Foundation Volume:	14.519 y ³
Foundation Result:	PASSED
Mount Twist:	0.551754 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Heath, TX 75032, USA
Wind Speed:	100 mph
Snow Load:	5 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.001649 ksf



Design Disclaimer

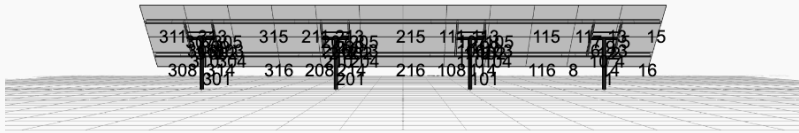
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

AutoDesigner Input

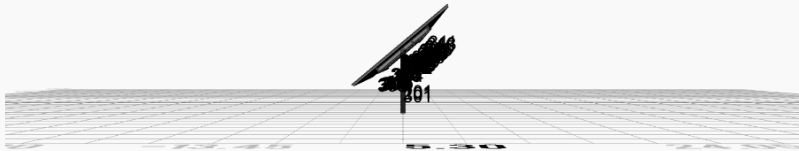
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Design Notes:

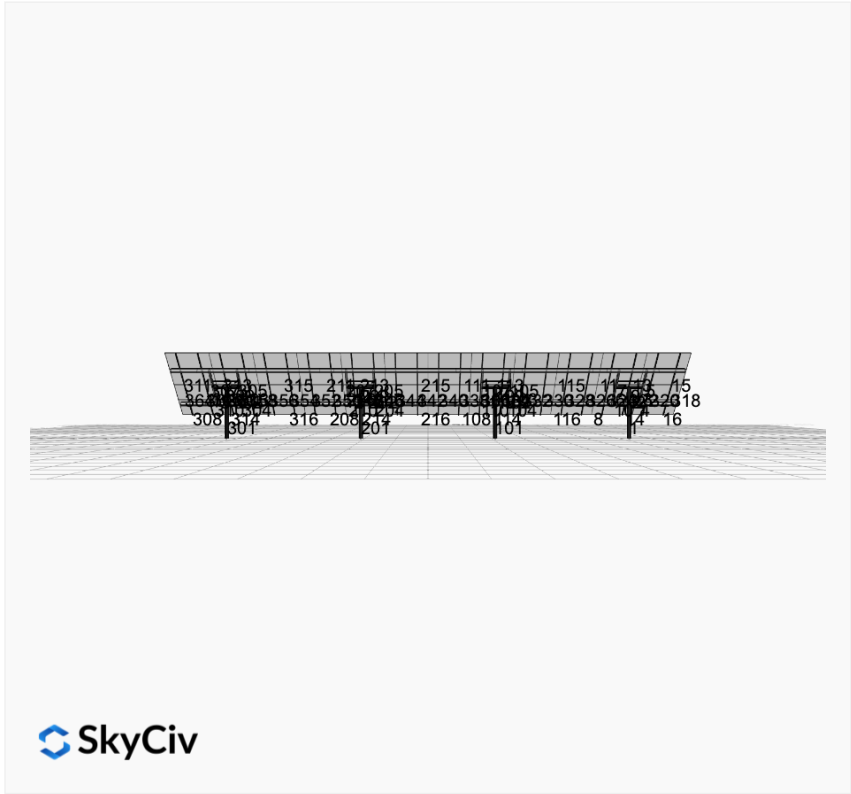
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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



 SkyCiv

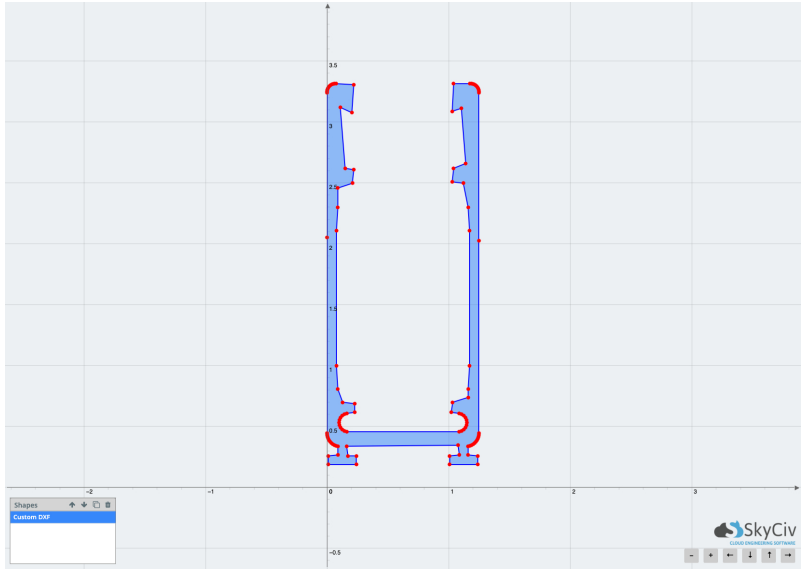


 SkyCiv



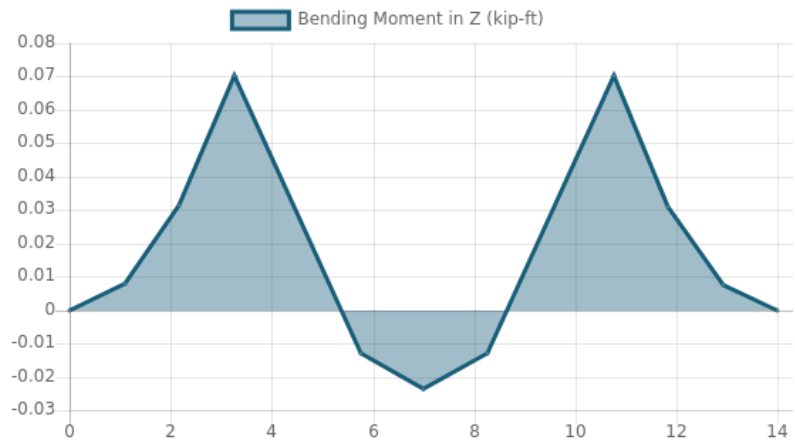
Rail Design Check

Rail Length: 14 ft
Additional Restraints Required: None
Tributary Width: 3.166666666666665 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0040 kip/ft
Snow (Y): -0.0034 kip/ft
Wind uplift Case A: 0.0879 kip/ft
Wind uplift Case A: 0.0879 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.1155 kip/ft

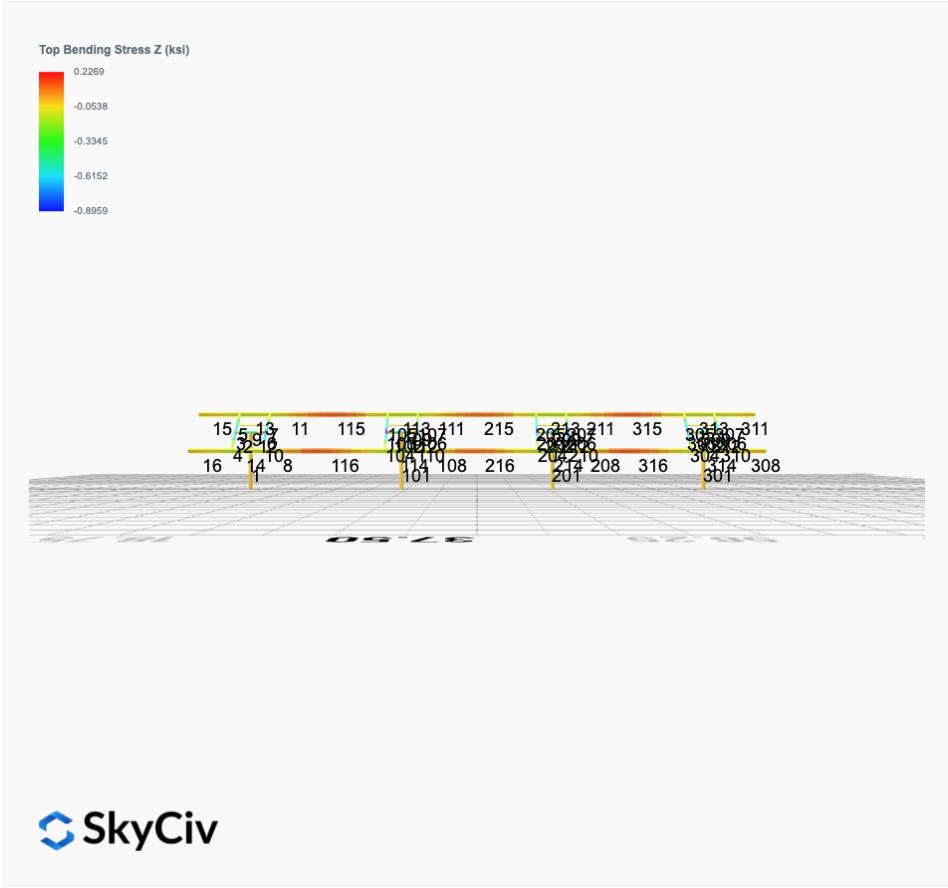
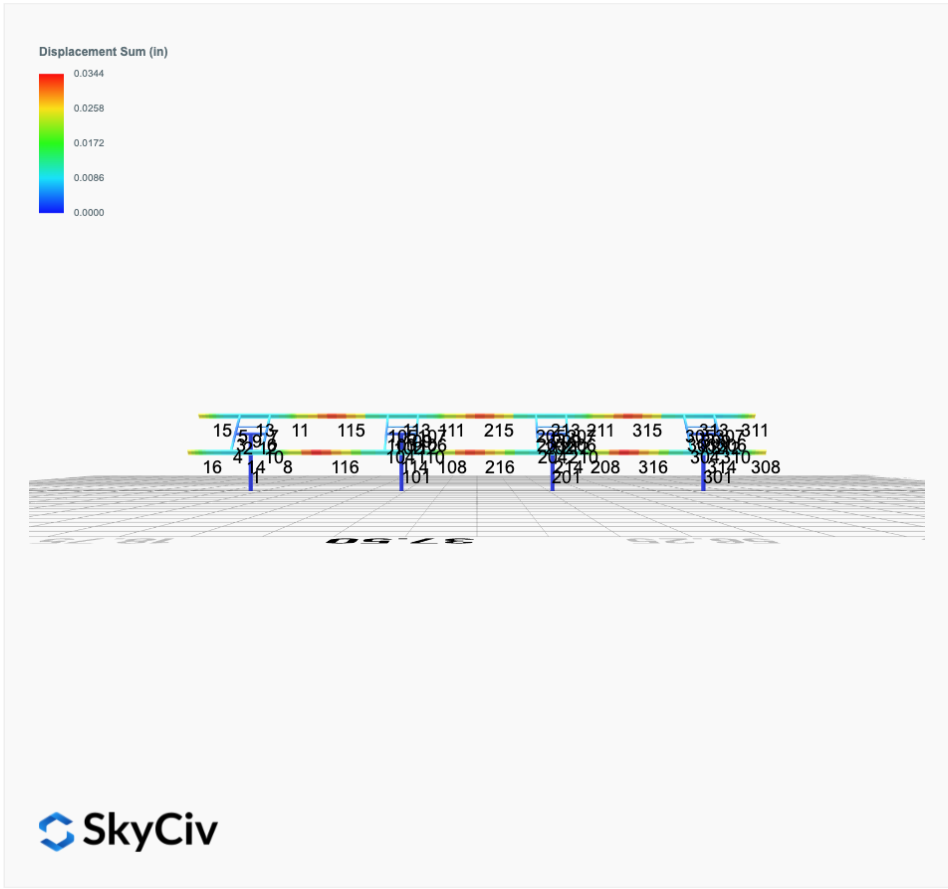


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	14.34776916	0.416	PASS
Material Yield	34.5	14.34776916	0.416	PASS
Material Strength	37	14.34776916	0.388	PASS

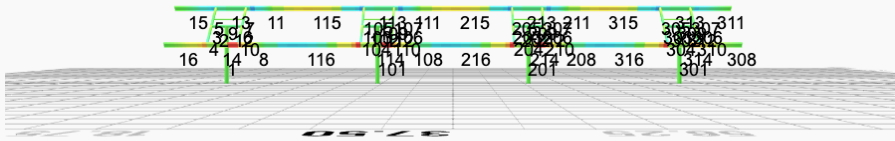
Member 1, ULS: 1. 1.4D



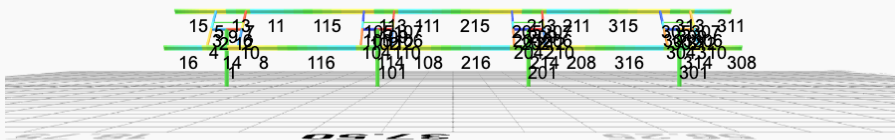
FEM Results (Envelope Worst Case for each member)



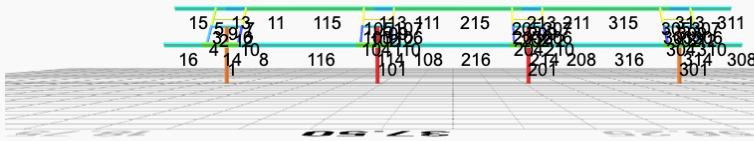
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0085	1.9297	0.0354	0.0701	-0.0088	-0.0359
ULS: 2. D + L	0.0085	1.9297	0.0354	0.0701	-0.0088	-0.0359
ULS: 3. D + (S or Lr or R)	0.0102	2.2348	0.0426	0.0844	-0.0106	-0.0480
ULS: 3. D + (S or Lr or R)	0.0085	1.9297	0.0354	0.0701	-0.0088	-0.0359
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0098	2.1586	0.0408	0.0808	-0.0101	-0.0450
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	1.9297	0.0354	0.0701	-0.0088	-0.0359
ULS: 5b. D + 0.7E	0.0085	1.9297	0.0354	0.0701	-0.0088	-0.0359
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0098	2.1586	0.0408	0.0808	-0.0101	-0.0450
ULS: 8. 0.6D + 0.7E	0.0051	1.1578	0.0212	0.0420	-0.0053	-0.0215
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2547	5.7980	0.1645	0.2960	-0.3319	25.6634
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.2547	5.7980	0.1645	0.2960	-0.3319	25.6634
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6068	-1.1522	-0.0638	-0.1035	0.2396	-19.0979
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.2253	-0.6915	-0.0622	-0.1001	0.2398	-23.9580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4376	5.0597	0.1377	0.2502	-0.2525	19.2295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4376	5.0597	0.1377	0.2502	-0.2525	19.2295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9585	-0.1529	-0.0336	-0.0494	0.1761	-14.3414
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6724	0.1926	-0.0324	-0.0469	0.1763	-17.9866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4389	4.8309	0.1323	0.2395	-0.2511	19.2386
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4389	4.8309	0.1323	0.2395	-0.2511	19.2386
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9572	-0.3817	-0.0390	-0.0601	0.1775	-14.3324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6711	-0.0362	-0.0378	-0.0576	0.1777	-17.9775
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2581	5.0261	0.1504	0.2680	-0.3284	25.6778
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.2581	5.0261	0.1504	0.2680	-0.3284	25.6778
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6035	-1.9241	-0.0780	-0.1315	0.2431	-19.0835
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.2219	-1.4634	-0.0764	-0.1282	0.2433	-23.9437

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	8.9155
Shear X	-5.4386
Shear Z	0.2622
Moment X	0.4693
Moment Y (Twist)	0.5516
Moment Z	43.0767

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	5.7980
Shear X	-3.2581
Shear Z	0.1645
Moment X	0.2960
Moment Y (Twist)	0.3319
Moment Z	25.6778

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	-0.0085	2.1587	-0.0021	-0.0041	0.0043	0.0841
ULS: 2. D + L	-0.0085	2.1587	-0.0021	-0.0041	0.0043	0.0841
ULS: 3. D + (S or Lr or R)	-0.0102	2.5103	-0.0025	-0.0049	0.0052	0.0965
ULS: 3. D + (S or Lr or R)	-0.0085	2.1587	-0.0021	-0.0041	0.0043	0.0841
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0098	2.4224	-0.0024	-0.0047	0.0050	0.0934
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0085	2.1587	-0.0021	-0.0041	0.0043	0.0841
ULS: 5b. D + 0.7E	-0.0085	2.1587	-0.0021	-0.0041	0.0043	0.0841

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0098	2.4224	-0.0024	-0.0047	0.0050	0.0934
ULS: 8. 0.6D + 0.7E	-0.0051	1.2952	-0.0012	-0.0024	0.0026	0.0505
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7570	6.6467	0.0045	0.0051	-0.0262	29.4074
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.7570	6.6467	0.0045	0.0051	-0.0262	29.4074
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9814	-1.4192	-0.0054	-0.0081	0.0230	-21.6307
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.5195	-0.8747	-0.0125	-0.0207	0.0438	-26.9310
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8212	5.7884	0.0025	0.0021	-0.0179	22.0858
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8212	5.7884	0.0025	0.0021	-0.0179	22.0858
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2327	-0.2610	-0.0048	-0.0077	0.0190	-16.1927
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.8862	0.1474	-0.0102	-0.0171	0.0346	-20.1679
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8199	5.5247	0.0029	0.0028	-0.0186	22.0766
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8199	5.5247	0.0029	0.0028	-0.0186	22.0766
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2340	-0.5248	-0.0045	-0.0071	0.0183	-16.2020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.8875	-0.1163	-0.0099	-0.0165	0.0339	-20.1772
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7536	5.7832	0.0053	0.0067	-0.0280	29.3738
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.7536	5.7832	0.0053	0.0067	-0.0280	29.3738
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9848	-2.2827	-0.0045	-0.0065	0.0213	-21.6644
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.5229	-1.7381	-0.0117	-0.0190	0.0421	-26.9646

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.2460
Shear X	-6.2581
Shear Z	-0.0206
Moment X	-0.0335
Moment Y (Twist)	0.0729
Moment Z	49.3422

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6467
Shear X	-3.7570
Shear Z	-0.0125
Moment X	-0.0207
Moment Y (Twist)	0.0438
Moment Z	29.4074

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	-0.0085	2.1587	0.0021	0.0041	-0.0043	0.0841
ULS: 2. D + L	-0.0085	2.1587	0.0021	0.0041	-0.0043	0.0841
ULS: 3. D + (S or Lr or R)	-0.0102	2.5103	0.0025	0.0049	-0.0052	0.0965
ULS: 3. D + (S or Lr or R)	-0.0085	2.1587	0.0021	0.0041	-0.0043	0.0841
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0098	2.4224	0.0024	0.0047	-0.0050	0.0934
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0085	2.1587	0.0021	0.0041	-0.0043	0.0841
ULS: 5b. D + 0.7E	-0.0085	2.1587	0.0021	0.0041	-0.0043	0.0841
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0098	2.4224	0.0024	0.0047	-0.0050	0.0934
ULS: 8. 0.6D + 0.7E	-0.0051	1.2952	0.0012	0.0024	-0.0026	0.0505
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.7570	6.6467	-0.0045	-0.0051	0.0262	29.4074
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.7570	6.6467	-0.0045	-0.0051	0.0262	29.4074
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.9814	-1.4192	0.0054	0.0081	-0.0230	-21.6307
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.5195	-0.8747	0.0125	0.0207	-0.0438	-26.9310
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8212	5.7884	-0.0025	-0.0021	0.0179	22.0858
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8212	5.7884	-0.0025	-0.0021	0.0179	22.0858
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2327	-0.2610	0.0048	0.0077	-0.0190	-16.1927
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.8862	0.1474	0.0102	0.0171	-0.0346	-20.1679

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8199	5.5247	-0.0029	-0.0028	0.0186	22.0766
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.8199	5.5247	-0.0029	-0.0028	0.0186	22.0766
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2340	-0.5248	0.0045	0.0071	-0.0183	-16.2020
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.8875	-0.1163	0.0099	0.0165	-0.0339	-20.1772
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7536	5.7832	-0.0053	-0.0067	0.0280	29.3738
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.7536	5.7832	-0.0053	-0.0067	0.0280	29.3738
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.9848	-2.2827	0.0045	0.0065	-0.0213	-21.6644
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.5229	-1.7381	0.0117	0.0190	-0.0421	-26.9646

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.2461
Shear X	-6.2581
Shear Z	0.0206
Moment X	0.0338
Moment Y (Twist)	0.0727
Moment Z	49.3422

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6467
Shear X	-3.7570
Shear Z	0.0125
Moment X	0.0207
Moment Y (Twist)	0.0438
Moment Z	29.4074

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	0.0085	1.9297	-0.0354	-0.0701	0.0088	-0.0359
ULS: 2. D + L	0.0085	1.9297	-0.0354	-0.0701	0.0088	-0.0359
ULS: 3. D + (S or Lr or R)	0.0102	2.2348	-0.0426	-0.0844	0.0106	-0.0480
ULS: 3. D + (S or Lr or R)	0.0085	1.9297	-0.0354	-0.0701	0.0088	-0.0359
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0098	2.1586	-0.0408	-0.0808	0.0102	-0.0450
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	1.9297	-0.0354	-0.0701	0.0088	-0.0359
ULS: 5b. D + 0.7E	0.0085	1.9297	-0.0354	-0.0701	0.0088	-0.0359
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0098	2.1586	-0.0408	-0.0808	0.0102	-0.0450
ULS: 8. 0.6D + 0.7E	0.0051	1.1578	-0.0212	-0.0420	0.0053	-0.0215
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.2547	5.7980	-0.1645	-0.2960	0.3319	25.6634
ULS: 5a. D + 0.6W_Wind downforce Case B only	-3.2547	5.7980	-0.1645	-0.2960	0.3319	25.6634
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.6068	-1.1522	0.0638	0.1035	-0.2396	-19.0979
ULS: 5a. D + 0.6W_Wind uplift Case B only	2.2253	-0.6915	0.0622	0.1001	-0.2398	-23.9580
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4376	5.0597	-0.1377	-0.2502	0.2525	19.2295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4376	5.0597	-0.1377	-0.2502	0.2525	19.2295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9585	-0.1529	0.0336	0.0494	-0.1761	-14.3414
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6724	0.1926	0.0324	0.0469	-0.1763	-17.9866
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.4389	4.8309	-0.1323	-0.2395	0.2511	19.2386
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-2.4389	4.8309	-0.1323	-0.2395	0.2511	19.2386
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9572	-0.3817	0.0390	0.0601	-0.1775	-14.3324
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	1.6711	-0.0362	0.0378	0.0576	-0.1777	-17.9775
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.2581	5.0261	-0.1504	-0.2680	0.3284	25.6778
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-3.2581	5.0261	-0.1504	-0.2680	0.3284	25.6778
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.6035	-1.9241	0.0780	0.1315	-0.2431	-19.0835
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	2.2219	-1.4634	0.0764	0.1282	-0.2433	-23.9437

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	8.9155
Shear X	-5.4386
Shear Z	-0.2622
Moment X	-0.4694
Moment Y (Twist)	0.5518
Moment Z	43.0774

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	5.7980
Shear X	-3.2581
Shear Z	-0.1645
Moment X	-0.2960
Moment Y (Twist)	0.3319
Moment Z	25.6778

Project Details

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

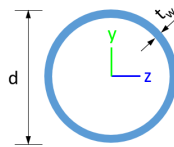


Design Input Information

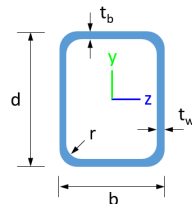
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

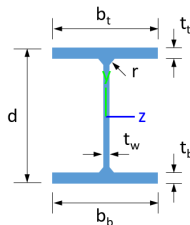
Section Dimensions



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				
8	6in Pipe Sch 80	6.63	0.43				



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
8	6in Pipe Sch 80	8.40	80.98	40.49	40.49	0.00	16.60	16.60

108	19	1.33	1.33	2.0 5	2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.09,2.33,2.09,2.09,2.09,2.35,2.09,2.09,2.10,1.98,2.0 9,2.09,2.09,1.50,2.09,2.09,2.09,2.39	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	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.64,1.67,1.67,1.65,1.71,1.6 7,1.67,1.65,1.81,1.67,1.67,1.66,1.65	3 0 0	2 0 0	1
111	19	1.33	1.33	2.0 5	2.09,2.09,2.09,2.09,2.09,2.09,2.06,2.06,2.06,2.04,2.06,2.06,2.06,2.05,2.07,2.07,1.89,1.91,2.0 7,2.07,1.98,1.96,2.06,2.06,2.06,2.06	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	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.06,1.05,1.05,1.05,1.06,1.05,1.05,1.06,1.08,1.0 5,1.05,1.05,1.07,1.05,1.05,1.05,1.06	3 0 0	2 0 0	1
114	19	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.04,1.05,1.05,1.05,1.04,1.04,1.04,1.05,1.69,1.0 4,1.04,1.05,1.08,1.05,1.05,1.05,1.04	3 0 0	2 0 0	1
115	19	6.63	6.63	10. 20	1.15,1.15,1.15,1.15,1.15,1.15,1.14,1.14,1.13,1.13,1.14,1.14,1.13,1.13,1.14,1.14,1.12,1.12,1.1 4,1.14,1.13,1.13,1.14,1.14,1.14,1.13	3 0 0	2 0 0	1
116	19	6.63	6.63	10. 20	1.15,1.15,1.15,1.15,1.15,1.15,1.16,1.16,1.16,1.21,1.16,1.16,1.16,1.20,1.16,1.16,1.16,1.12,1.1 6,1.16,1.16,1.15,1.16,1.16,1.16,1.19	3 0 0	2 0 0	1
201	8	15.7 5	15.7 5	7.5 0	-	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	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.17,1.17,1.1 8,1.18,1.17,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1
204	16	2.44	2.44	3.7 5	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.64,1.67,1.67,1.65,1.71,1.6 7,1.67,1.65,1.81,1.67,1.67,1.66,1.65	3 0 0	2 0 0	1
205	16	1.52	1.52	2.3 3	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.6 7,1.67,1.66,1.66,1.67,1.67,1.66,1.67	3 0 0	2 0 0	1
206	16	0.92	0.92	1.4 2	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.18,1.18,1.18,1.17,1.17,1.1 8,1.18,1.17,1.17,1.18,1.18,1.18,1.18	3 0 0	2 0 0	1
207	16	1.52	1.52	2.3 3	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.6 7,1.67,1.66,1.66,1.67,1.67,1.66,1.67	3 0 0	2 0 0	1
208	19	1.33	1.33	2.0 5	2.07,2.07,2.07,2.07,2.07,2.07,2.08,2.08,2.08,2.33,2.08,2.08,2.08,2.35,2.08,2.08,2.08,1.66,2.0 8,2.08,2.08,1.22,2.08,2.08,2.08,2.24	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	1.68,1.68,1.68,1.68,1.68,1.68,1.67,1.67,1.66,1.64,1.67,1.67,1.66,1.64,1.67,1.67,1.65,1.71,1.6 7,1.67,1.65,1.82,1.67,1.67,1.66,1.65	3 0 0	2 0 0	1
211	19	1.33	1.33	2.0 5	2.07,2.07,2.07,2.07,2.07,2.07,1.94,1.94,1.86,1.81,1.93,1.93,1.86,1.81,1.98,1.98,1.70,1.70,1.9 7,1.97,1.77,1.75,1.93,1.93,1.88,1.83	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	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.06,1.05,1.05,1.05,1.06,1.05,1.05,1.06,1.08,1.0 5,1.05,1.05,1.07,1.05,1.05,1.05,1.06	3 0 0	2 0 0	1
214	19	4.88	4.00	7.5 0	1.04,1.04,1.04,1.04,1.04,1.04,1.05,1.05,1.05,1.04,1.05,1.05,1.05,1.04,1.04,1.04,1.05,1.69,1.0 4,1.04,1.05,1.08,1.05,1.05,1.05,1.04	3 0 0	2 0 0	1
215	19	6.63	6.63	10. 20	1.16,1.16,1.16,1.16,1.16,1.16,1.15,1.15,1.14,1.14,1.15,1.15,1.14,1.14,1.15,1.15,1.14,1.14,1.1 5,1.15,1.14,1.14,1.15,1.15,1.14,1.14	3 0 0	2 0 0	1
216	19	6.63	6.63	10. 20	1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.16,1.19,1.16,1.16,1.16,1.19,1.16,1.16,1.16,1.14,1.1 6,1.16,1.16,1.11,1.16,1.16,1.16,1.18	3 0 0	2 0 0	1
301	8	15.7 5	15.7 5	7.5 0	-	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	23.91	6.12	40.24	43.62
114	133.20	104.94	23.87	6.12	40.24	43.62
115	133.20	69.16	17.37	6.12	40.24	43.62
116	133.20	69.16	17.29	6.12	40.24	43.62
201	378.22	219.95	62.23	62.23	113.47	113.47
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	23.91	6.12	40.24	43.62
214	133.20	104.94	23.84	6.12	40.24	43.62
215	133.20	69.16	17.57	6.12	40.24	43.62
216	133.20	69.16	17.19	6.12	40.24	43.62
301	378.22	219.95	62.23	62.23	113.47	113.47
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	52.83	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	52.83	32.87	6.12	40.24	43.62
312	198.33	182.14	21.95	21.95	59.50	59.50
313	133.20	104.94	25.18	6.12	40.24	43.62
314	133.20	104.94	23.88	6.12	40.24	43.62
315	133.20	69.16	17.37	6.12	40.24	43.62
316	133.20	69.16	16.91	6.12	40.24	43.62

Design Ratio

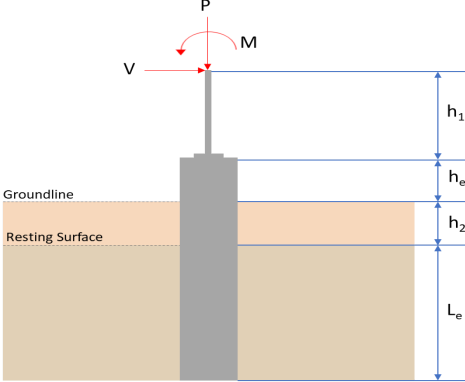
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	φ	Status
1	0.041	0.692	0.024	0.048	0.002	0.720	#13	0.431	Not Required	Pass
2	0.001	0.303	0.220	0.068	0.044	0.524	#13	0.114	Not Required	Pass
3	0.003	0.581	0.017	0.058	0.002	0.591	#13	0.045	Not Required	Pass
4	0.002	0.544	0.050	0.055	0.012	0.588	#13	0.080	Not Required	Pass

4	0.003	0.360	0.044	0.058	0.011	0.364	#13	0.074	Not Required	Pass
5	0.003	0.360	0.044	0.058	0.011	0.364	#13	0.074	Not Required	Pass
6	0.004	0.656	0.032	0.067	0.007	0.689	#13	0.045	Not Required	Pass
7	0.004	0.407	0.063	0.065	0.016	0.423	#13	0.074	Not Required	Pass
8	0.001	0.078	0.058	0.041	0.006	0.104	#13	0.095	Not Required	Pass
9	0.004	0.054	0.055	0.002	0.001	0.110	#13	0.204	Not Required	Pass
10	0.004	0.600	0.064	0.060	0.014	0.616	#13	0.080	Not Required	Pass
11	0.001	0.077	0.058	0.045	0.006	0.100	#13	0.095	Not Required	Pass
12	0.001	0.371	0.245	0.078	0.048	0.617	#13	0.053	Not Required	Pass
13	0.002	0.200	0.145	0.058	0.008	0.271	#13	0.286	Not Required	Pass
14	0.003	0.186	0.145	0.053	0.008	0.256	#13	0.190	Not Required	Pass
15	0.000	0.064	0.049	0.028	0.004	0.106	#13	Not Required	Not Required	Pass
16	0.000	0.060	0.049	0.026	0.004	0.102	#13	Not Required	Not Required	Pass
101	0.047	0.793	0.002	0.055	0.000	0.816	#13	0.431	Not Required	Pass
102	0.001	0.388	0.268	0.084	0.052	0.656	#13	0.035	Not Required	Pass
103	0.004	0.703	0.025	0.070	0.004	0.727	#13	0.045	Not Required	Pass
104	0.004	0.670	0.062	0.067	0.013	0.710	#13	0.080	Not Required	Pass
105	0.004	0.436	0.063	0.070	0.016	0.450	#13	0.074	Not Required	Pass
106	0.004	0.711	0.025	0.071	0.005	0.733	#13	0.045	Not Required	Pass
107	0.004	0.442	0.060	0.071	0.015	0.456	#13	0.074	Not Required	Pass
108	0.002	0.050	0.053	0.042	0.006	0.084	#13	0.095	Not Required	Pass
109	0.005	0.056	0.049	0.001	0.000	0.106	#13	0.204	Not Required	Pass
110	0.004	0.667	0.059	0.067	0.013	0.702	#13	0.080	Not Required	Pass
111	0.001	0.063	0.054	0.045	0.006	0.080	#13	0.095	Not Required	Pass
112	0.001	0.390	0.270	0.084	0.053	0.660	#13	0.035	Not Required	Pass
113	0.002	0.213	0.146	0.059	0.008	0.318	#13	0.286	Not Required	Pass
114	0.004	0.222	0.147	0.056	0.008	0.330	#13	0.286	Not Required	Pass
115	0.002	0.276	0.077	0.046	0.006	0.346	#13	0.473	Not Required	Pass
116	0.002	0.256	0.078	0.044	0.006	0.325	#13	0.473	Not Required	Pass
201	0.047	0.793	0.002	0.055	0.000	0.816	#13	0.431	Not Required	Pass
202	0.001	0.390	0.270	0.084	0.053	0.660	#13	0.035	Not Required	Pass
203	0.004	0.711	0.025	0.072	0.005	0.733	#13	0.045	Not Required	Pass
204	0.004	0.667	0.059	0.067	0.013	0.702	#13	0.080	Not Required	Pass
205	0.004	0.442	0.060	0.071	0.015	0.456	#13	0.074	Not Required	Pass
206	0.004	0.703	0.025	0.070	0.004	0.727	#13	0.045	Not Required	Pass
207	0.004	0.436	0.063	0.070	0.016	0.450	#13	0.074	Not Required	Pass
208	0.001	0.059	0.059	0.044	0.006	0.092	#13	0.095	Not Required	Pass
209	0.005	0.056	0.049	0.001	0.000	0.106	#13	0.204	Not Required	Pass
210	0.004	0.670	0.062	0.067	0.013	0.710	#13	0.080	Not Required	Pass
211	0.001	0.073	0.059	0.046	0.006	0.082	#13	0.095	Not Required	Pass
212	0.001	0.388	0.268	0.084	0.052	0.656	#13	0.035	Not Required	Pass
213	0.002	0.213	0.146	0.059	0.008	0.318	#13	0.286	Not Required	Pass
214	0.004	0.222	0.147	0.056	0.008	0.330	#13	0.286	Not Required	Pass
215	0.002	0.254	0.077	0.045	0.006	0.323	#13	0.473	Not Required	Pass
216	0.003	0.221	0.078	0.042	0.006	0.289	#13	0.473	Not Required	Pass
301	0.041	0.692	0.024	0.048	0.002	0.720	#13	0.431	Not Required	Pass
302	0.001	0.371	0.245	0.078	0.048	0.617	#13	0.053	Not Required	Pass
303	0.004	0.656	0.032	0.067	0.007	0.689	#13	0.045	Not Required	Pass
304	0.004	0.600	0.064	0.060	0.014	0.616	#13	0.080	Not Required	Pass
305	0.004	0.407	0.063	0.065	0.016	0.423	#13	0.074	Not Required	Pass
306	0.003	0.581	0.017	0.058	0.002	0.591	#13	0.045	Not Required	Pass
307	0.003	0.360	0.044	0.058	0.011	0.365	#13	0.074	Not Required	Pass
308	0.000	0.060	0.049	0.026	0.004	0.102	#13	Not Required	Not Required	Pass
309	0.004	0.054	0.055	0.002	0.001	0.110	#13	0.204	Not Required	Pass

310	0.003	0.544	0.050	0.055	0.012	0.588	#13	0.080	Not Required	Pass
311	0.000	0.064	0.049	0.028	0.004	0.106	#13	Not Required	Not Required	Pass
312	0.001	0.303	0.220	0.068	0.044	0.524	#13	0.114	Not Required	Pass
313	0.002	0.200	0.145	0.058	0.008	0.271	#13	0.190	Not Required	Pass
314	0.003	0.186	0.145	0.053	0.008	0.256	#13	0.286	Not Required	Pass
315	0.002	0.283	0.078	0.045	0.006	0.347	#13	0.473	Not Required	Pass
316	0.002	0.262	0.078	0.041	0.006	0.331	#13	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 Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.798</td> <td>8.915</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.258</td> <td>-5.439</td> </tr> <tr> <td>V_z (kip)</td> <td>0.165</td> <td>0.262</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.296</td> <td>0.469</td> </tr> <tr> <td>M_z (kipft)</td> <td>25.678</td> <td>43.077</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)	5.798	8.915	V_x (kip)	-3.258	-5.439	V_z (kip)	0.165	0.262	M_x (kipft)	0.296	0.469	M_z (kipft)	25.678	43.077	
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M_z (kipft)	25.678	43.077																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.258 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.51879 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.678 \text{ kipft}) + ((-3.258 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0889 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.4131 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.165 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.026274 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.296 \text{ kipft}) + ((0.165 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.047134 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.8898 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4131 \text{ ft}), (1.8898 \text{ ft})]$$

$$L_{e,req} = 5.413 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.413 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.90217$$

Status: **PASS**
Ratio: **0.900**

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

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

$$q = \frac{(5.798 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36238 \text{ kip/ft}^2$$

$$q = 0.00238 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36238 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18119$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.51879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0889 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0889 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.0889 \text{ kipft/ft})) + (4 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1683 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0889 \text{ kipft/ft})) + (3 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (4.0889 \text{ kipft/ft})) + (2 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.16981 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0889 \text{ kipft/ft})) + ((-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.84416 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1683 \text{ ft})}{2}$$

$$p_a = 0.31262 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16981 \text{ kip/ft}^2)}{(0.31262 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.54319$$

p_a - Allowable lateral soil pressure at depth L_e ,

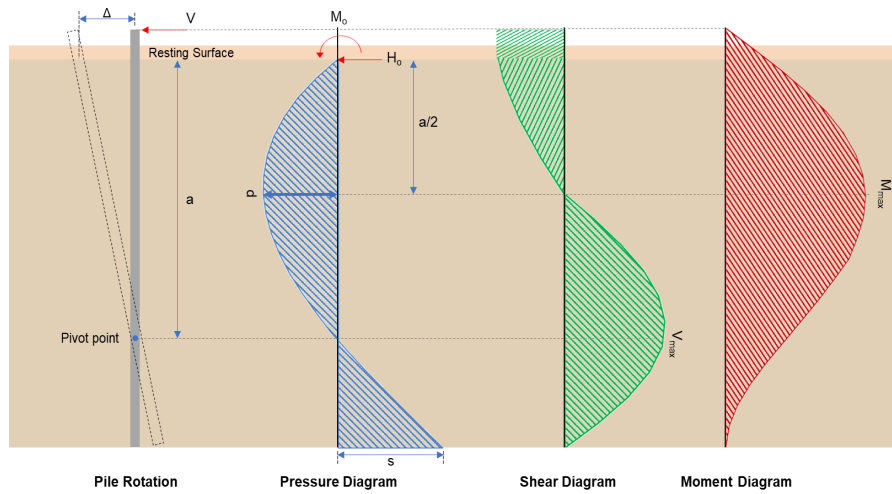
Status: **PASS**
Ratio: **0.540**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.84416 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93796$	<p>Status: PASS Ratio: 0.940</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.026274 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.047134 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.047134 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.026274 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.047134 \text{ kipft/ft})) + (4 \times (0.026274 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3452 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.047134 \text{ kipft/ft})) + (3 \times (0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.047134 \text{ kipft/ft})) + (2 \times (0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = 0.01996 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.047134 \text{ kipft/ft})) + ((0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = 0.041985 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3452 \text{ ft})}{2}$ $p_a = 0.32589 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.01996 \text{ kip/ft}^2)}{(0.32589 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.061247$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.060</p>

$$\text{Ratio} = \frac{(0.041985 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.04665$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.439 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.86608 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.077 \text{ kipft}) + ((-5.439 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8594 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8594 \text{ kipft/ft})}{(-0.86608 \text{ kip/ft})}$$

$$E = 7.92 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8594 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.86608 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.8594 \text{ kipft/ft})) + (4 \times (-0.86608 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1678 \text{ ft}$$

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

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

$$V_{max} = ((-0.86608 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1678 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1678 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.376 \text{ kip}$$

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

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

$$M_{max} = ((-0.86608 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(7.92 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 29.249 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.262 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.04172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.469 \text{ kipft}) + ((0.262 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.074682 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.074682 \text{ kipft/ft})}{(0.04172 \text{ kip/ft})}$$

$$E = 1.7901 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.074682 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.04172 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.074682 \text{ kipft/ft})) + (4 \times (0.04172 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3454 \text{ ft}$$

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

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

$$V_{max} = ((0.04172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3454 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3454 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20017 \text{ kip}$$

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

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

$$M_{max} = ((0.04172 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(1.7901 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.51178 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.915 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.3 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.3 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(8.915 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0033325$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.915 \text{ kip} \rightarrow 8915 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8915 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.67 \text{ kip}$$

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

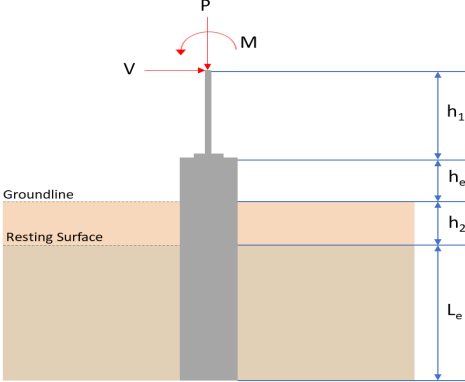
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.67 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.67 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.67 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.87 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.376 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.376 \text{ kip})}{(110.87 \text{ kip})}$ $\text{Ratio} = 0.093591$ <p>Considering z-direction:</p> <p>$V_{max} = 0.20017 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.20017 \text{ kip})}{(110.87 \text{ kip})}$ $\text{Ratio} = 0.0018055$	<p>Status: PASS Ratio: 0.090</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: $M_{max} = 29.249 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(29.249 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11718$	<p>Status: PASS Ratio: 0.120</p>
	<p>Considering z-direction: $M_{max} = 0.51178 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.51178 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0020504$	<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 $L = 6$ 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>5.798</td> <td>8.915</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.258</td> <td>-5.439</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.165</td> <td>-0.262</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.296</td> <td>-0.469</td> </tr> <tr> <td>M_z (kipft)</td> <td>25.678</td> <td>43.077</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)	5.798	8.915	V_x (kip)	-3.258	-5.439	V_z (kip)	-0.165	-0.262	M_x (kipft)	-0.296	-0.469	M_z (kipft)	25.678	43.077	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.258 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.51879 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.678 \text{ kipft}) + ((-3.258 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0889 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.4131 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.165 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026274 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.296 \text{ kipft}) + ((-0.165 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.047134 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2252 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.4131 \text{ ft}), (1.2252 \text{ ft})]$$

$$L_{e,req} = 5.413 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.413 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.90217$$

Status: **PASS**
Ratio: **0.900**

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

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

$$q = \frac{(5.798 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36238 \text{ kip/ft}^2$$

$$q = 0.00238 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36238 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18119$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.51879 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.0889 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0889 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.0889 \text{ kipft/ft})) + (4 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1683 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.0889 \text{ kipft/ft})) + (3 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (4.0889 \text{ kipft/ft})) + (2 \times (-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.16981 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.0889 \text{ kipft/ft})) + ((-0.51879 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.84416 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.1683 \text{ ft})}{2}$$

$$p_a = 0.31262 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16981 \text{ kip/ft}^2)}{(0.31262 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.54319$$

p_a - Allowable lateral soil pressure at depth L_e ,

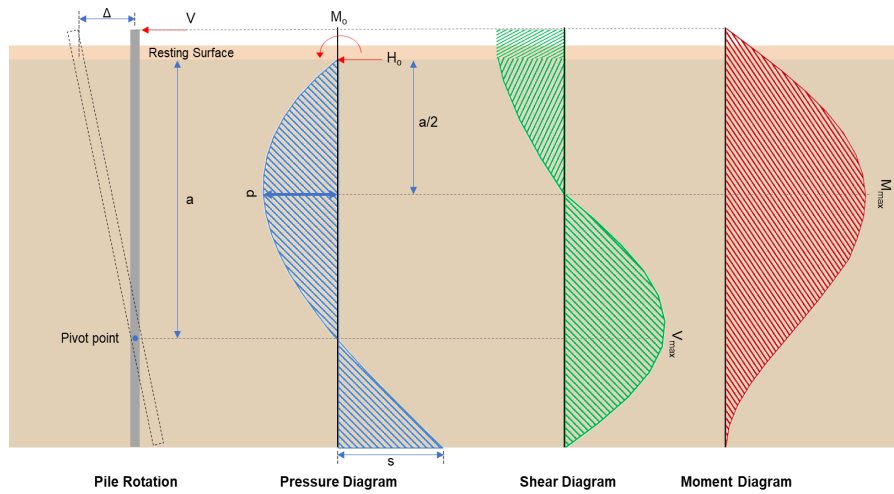
Status: **PASS**
Ratio: **0.540**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.84416 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.93796$	Status: PASS Ratio: 0.940
	<p>Considering z-direction:</p> <p>$H_o = -0.026274 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.047134 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.047134 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.026274 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.047134 \text{ kipft/ft})) + (4 \times (-0.026274 \text{ kip/ft}) \times (6 \text{ ft}))}$ $a = 4.3452 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.047134 \text{ kipft/ft})) + (3 \times (-0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.047134 \text{ kipft/ft})) + (2 \times (-0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]}$ $p = -0.0096903 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.047134 \text{ kipft/ft})) + ((-0.026274 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$ $s = -0.010563 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.3452 \text{ ft})}{2}$ $p_a = 0.32589 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.0096903 \text{ kip/ft}^2)}{(0.32589 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.029735$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$ $p_s = 0.9 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: -0.030

$$\text{Ratio} = \frac{(-0.010563 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.011736$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-5.439 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.86608 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(43.077 \text{ kipft}) + ((-5.439 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.8594 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.8594 \text{ kipft/ft})}{(-0.86608 \text{ kip/ft})}$$

$$E = 7.92 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.8594 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.86608 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (6.8594 \text{ kipft/ft})) + (4 \times (-0.86608 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.1678 \text{ ft}$$

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

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

$$V_{max} = ((-0.86608 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1678 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1678 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.376 \text{ kip}$$

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

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

$$M_{max} = ((-0.86608 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(7.92 \text{ ft})}{(6 \text{ ft})} + \frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (7.92 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.1678 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 29.249 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.262 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.04172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.469 \text{ kipft}) + ((-0.262 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.074682 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.074682 \text{ kipft/ft})}{(-0.04172 \text{ kip/ft})}$$

$$E = 1.7901 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.074682 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.04172 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.074682 \text{ kipft/ft})) + (4 \times (-0.04172 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.3454 \text{ ft}$$

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

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

$$V_{max} = ((-0.04172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3454 \text{ ft})}{(6 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3454 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.20017 \text{ kip}$$

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

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

$$M_{max} = ((-0.04172 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(1.7901 \text{ ft})}{(6 \text{ ft})} + \frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (1.7901 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.3454 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.51178 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.915 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.3 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.3 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(8.915 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0033325$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.915 \text{ kip} \rightarrow 8915 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8915 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.67 \text{ kip}$$

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

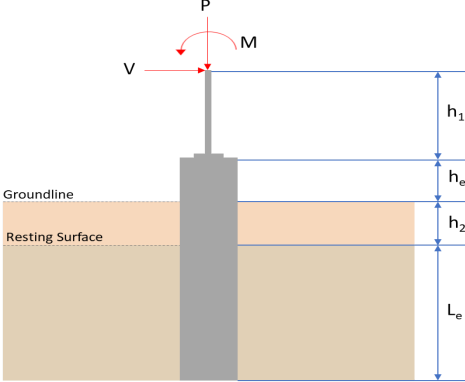
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.67 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.67 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.67 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.87 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.376 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.376 \text{ kip})}{(110.87 \text{ kip})}$ $\text{Ratio} = 0.093591$ <p>Considering z-direction:</p> <p>$V_{max} = 0.20017 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.20017 \text{ kip})}{(110.87 \text{ kip})}$ $\text{Ratio} = 0.0018055$	<p>Status: PASS Ratio: 0.090</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: $M_{max} = 29.249 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(29.249 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.11718$	<p>Status: PASS Ratio: 0.120</p>
	<p>Considering z-direction: $M_{max} = 0.51178 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.51178 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0020504$	<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 $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.647</td> <td>10.246</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.757</td> <td>-6.258</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.013</td> <td>-0.021</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.021</td> <td>-0.034</td> </tr> <tr> <td>M_z (kipft)</td> <td>29.407</td> <td>49.342</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)	6.647	10.246	V_x (kip)	-3.757	-6.258	V_z (kip)	-0.013	-0.021	M_x (kipft)	-0.021	-0.034	M_z (kipft)	29.407	49.342	
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M_z (kipft)	29.407	49.342																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.757 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.59825 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(29.407 \text{ kipft}) + ((-3.757 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.6826 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5845 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0020701 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.021 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0033439 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.58005 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[5.5845 \text{ ft}, 0.58005 \text{ ft}]$$

$$L_{e,req} = 5.584 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.584 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.89344$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.647 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.41544 \text{ kip/ft}^2$$

$$q = 0.41544 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.41544 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20772$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59825 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.6826 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.6826 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.6826 \text{ kipft/ft})) + (4 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3476 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.6826 \text{ kipft/ft})) + (3 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.6826 \text{ kipft/ft})) + (2 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.16498 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.6826 \text{ kipft/ft})) + ((-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86419 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3476 \text{ ft})}{2}$$

$$p_a = 0.32607 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16498 \text{ kip/ft}^2)}{(0.32607 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.50595$$

p_a - Allowable lateral soil pressure at depth L_e ,

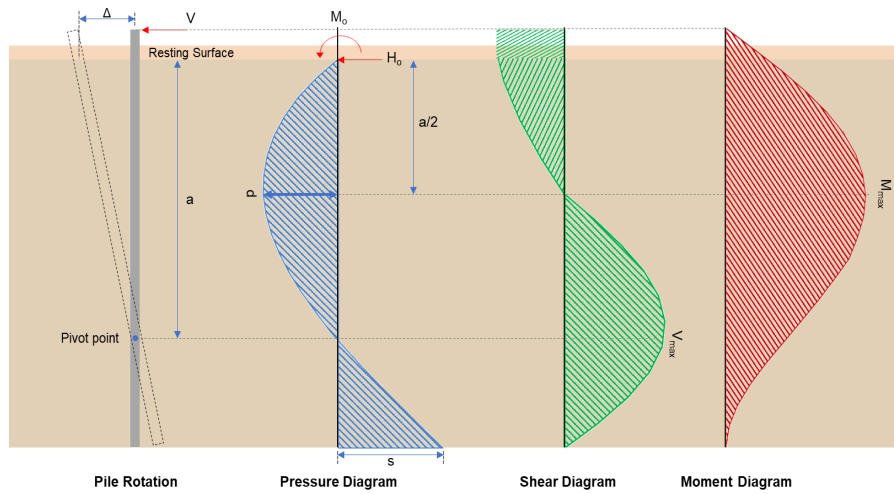
Status: **PASS**
Ratio: **0.510**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.86419 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9218$	Status: PASS Ratio: 0.920
	<p>Considering z-direction:</p> <p>$H_o = -0.0020701 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0033439 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0033439 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0033439 \text{ kipft/ft})) + (4 \times (-0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.542 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0033439 \text{ kipft/ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0033439 \text{ kipft/ft})) + (2 \times (-0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = -0.00078415 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0033439 \text{ kipft/ft})) + ((-0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = -0.00096 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.542 \text{ ft})}{2}$ $p_a = 0.34065 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.00078415 \text{ kip/ft}^2)}{(0.34065 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.0023019$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$\text{Ratio} = \frac{(-0.00096 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.001024$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.258 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.9965 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.342 \text{ kipft}) + ((-6.258 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.857 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.857 \text{ kipft/ft})}{(-0.9965 \text{ kip/ft})}$$

$$E = 7.8846 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.857 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.9965 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.857 \text{ kipft/ft})) + (4 \times (-0.9965 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3467 \text{ ft}$$

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

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

$$V_{max} = ((-0.9965 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3467 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3467 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.527 \text{ kip}$$

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

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

$$M_{max} = ((-0.9965 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(7.8846 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 33.769 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.021 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0033439 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034 \text{ kipft}) + ((-0.021 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.005414 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.005414 \text{ kipft/ft})}{(-0.0033439 \text{ kip/ft})}$$

$$E = 1.619 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.005414 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0033439 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.005414 \text{ kipft/ft})) + (4 \times (-0.0033439 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5418 \text{ ft}$$

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

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

$$V_{max} = ((-0.0033439 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5418 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5418 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.015133 \text{ kip}$$

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

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

$$M_{max} = ((-0.0033439 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(1.619 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.039892 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.246 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.256 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.256 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(10.246 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.00383$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.246 \text{ kip} \rightarrow 10246 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10246 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.85 \text{ kip}$$

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

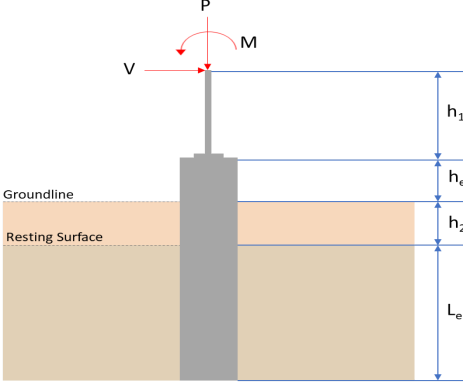
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.85 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.85 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.85 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.98 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.527 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.527 \text{ kip})}{(110.98 \text{ kip})}$ $\text{Ratio} = 0.10386$ <p>Considering z-direction:</p> <p>$V_{max} = 0.015133 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.015133 \text{ kip})}{(110.98 \text{ kip})}$ $\text{Ratio} = 0.00013635$	<p>Status: PASS Ratio: 0.100</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: $M_{max} = 33.769 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(33.769 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13529$	<p>Status: PASS Ratio: 0.140</p>
	<p>Considering z-direction: $M_{max} = 0.039892 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.039892 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00015982$	<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 $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.647</td> <td>10.246</td> </tr> <tr> <td>V_x (kip)</td> <td>-3.757</td> <td>-6.258</td> </tr> <tr> <td>V_z (kip)</td> <td>0.013</td> <td>0.021</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.021</td> <td>0.034</td> </tr> <tr> <td>M_z (kipft)</td> <td>29.407</td> <td>49.342</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)	6.647	10.246	V_x (kip)	-3.757	-6.258	V_z (kip)	0.013	0.021	M_x (kipft)	0.021	0.034	M_z (kipft)	29.407	49.342	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-3.757 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.59825 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(29.407 \text{ kipft}) + ((-3.757 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.6826 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.5845 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0020701 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.021 \text{ kipft}) + ((0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0033439 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.70817 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[5.5845 \text{ ft}, 0.70817 \text{ ft}]$$

$$L_{e,req} = 5.584 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.584 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.89344$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.647 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.41544 \text{ kip/ft}^2$$

$$q = 0.41544 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.41544 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20772$$

Status: **PASS**
Ratio: **0.210**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.59825 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.6826 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.6826 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.6826 \text{ kipft/ft})) + (4 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3476 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.6826 \text{ kipft/ft})) + (3 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.6826 \text{ kipft/ft})) + (2 \times (-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.16498 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.6826 \text{ kipft/ft})) + ((-0.59825 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86419 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3476 \text{ ft})}{2}$$

$$p_a = 0.32607 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.16498 \text{ kip/ft}^2)}{(0.32607 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.50595$$

p_a - Allowable lateral soil pressure at depth L_e ,

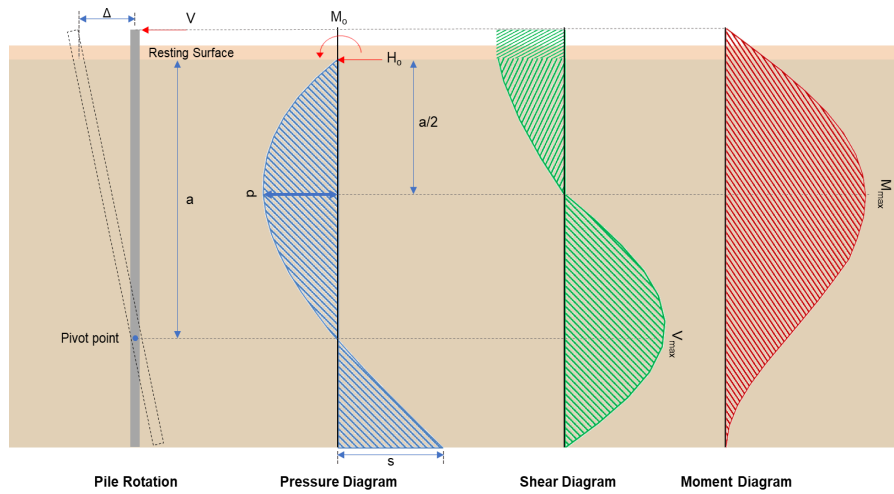
Status: **PASS**
Ratio: **0.510**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.86419 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9218$	<p>Status: PASS Ratio: 0.920</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.0020701 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0033439 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0033439 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0033439 \text{ kipft/ft})) + (4 \times (0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.542 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0033439 \text{ kipft/ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0033439 \text{ kipft/ft})) + (2 \times (0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.0014564 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0033439 \text{ kipft/ft})) + ((0.0020701 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.0030145 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.542 \text{ ft})}{2}$ $p_a = 0.34065 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0014564 \text{ kip/ft}^2)}{(0.34065 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0042754$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.000</p>

$$Ratio = \frac{(0.0030145 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0032155$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.258 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.9965 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(49.342 \text{ kipft}) + ((-6.258 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.857 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.857 \text{ kipft/ft})}{(-0.9965 \text{ kip/ft})}$$

$$E = 7.8846 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.857 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.9965 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.857 \text{ kipft/ft})) + (4 \times (-0.9965 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3467 \text{ ft}$$

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

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

$$V_{max} = ((-0.9965 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3467 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3467 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 11.527 \text{ kip}$$

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

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

$$M_{max} = ((-0.9965 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(7.8846 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (7.8846 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3467 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 33.769 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.021 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0033439 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.034 \text{ kipft}) + ((0.021 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.005414 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.005414 \text{ kipft/ft})}{(0.0033439 \text{ kip/ft})}$$

$$E = 1.619 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.005414 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0033439 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.005414 \text{ kipft/ft})) + (4 \times (0.0033439 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.5418 \text{ ft}$$

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

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

$$V_{max} = ((0.0033439 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5418 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5418 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.015133 \text{ kip}$$

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

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

$$M_{max} = ((0.0033439 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(1.619 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (1.619 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.5418 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.039892 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.246 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.256 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.256 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(10.246 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.00383$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.246 \text{ kip} \rightarrow 10246 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(10246 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.85 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.85 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.85 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.85 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.98 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 11.527 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(11.527 \text{ kip})}{(110.98 \text{ kip})}$ $\text{Ratio} = 0.10386$ <p>Considering z-direction:</p> <p>$V_{max} = 0.015133 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.015133 \text{ kip})}{(110.98 \text{ kip})}$ $\text{Ratio} = 0.00013635$	<p>Status: PASS Ratio: 0.100</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: $M_{max} = 33.769 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(33.769 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.13529$	<p>Status: PASS Ratio: 0.140</p>
	<p>Considering z-direction: $M_{max} = 0.039892 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.039892 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00015982$	<p>Status: PASS Ratio: 0.000</p>