

Your Project Calculations



Project Name: MTSOLAR_10AG1252B7212-JB-RevA

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=MTSOLAR_10AG1252B7212-JB-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/7_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=amhuFK1mu3SgSikfGNGTM3NFwpvg1eaCdCCGjdPpcM76mM8zGO0NFdklLwtX8B2a

Array Specification

Product:	Beam
Unique ID:	5P-19.75-8TOP-SD-72-L-5Hx13W-6GA3
Duty Classification:	SD
Module Width:	44.00 in
Module Length:	90.00in
Number of Rows:	5
Number of Columns:	13
Total Number of Modules:	65
Desired Tilt Angle:	55
Front Edge Clearance:	6
Total Array Height at Tilt:	21.10 ft
Total Frame Length:	98.50 ft
Frame Weight:	4480 lbs
Array Dimensions N/S:	18.54 ft
Array Dimensions E/W:	98.58 ft
Rail Length:	222.50 in
Rail Spacing:	3.75 ft
Rail Check:	Not Checked

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	13.59 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 4.50 ft Pile 2: 4.50 ft Pile 3: 4.50 ft Pile 4: 4.50 ft Pile 5: 4.50 ft
Foundation Volume:	13.333 y ³
Foundation Result:	PASSED
Mount Twist:	0.803207 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sedimentary
Site Location:	Sherman, ME, USA
Wind Speed:	75 mph
Snow Load:	60 psf
Design Uplift Pressure:	0.008660 ksf
Design Downforce Pressure:	-0.008660 ksf
Design Snow Pressure:	0.009897 ksf



Design Disclaimer

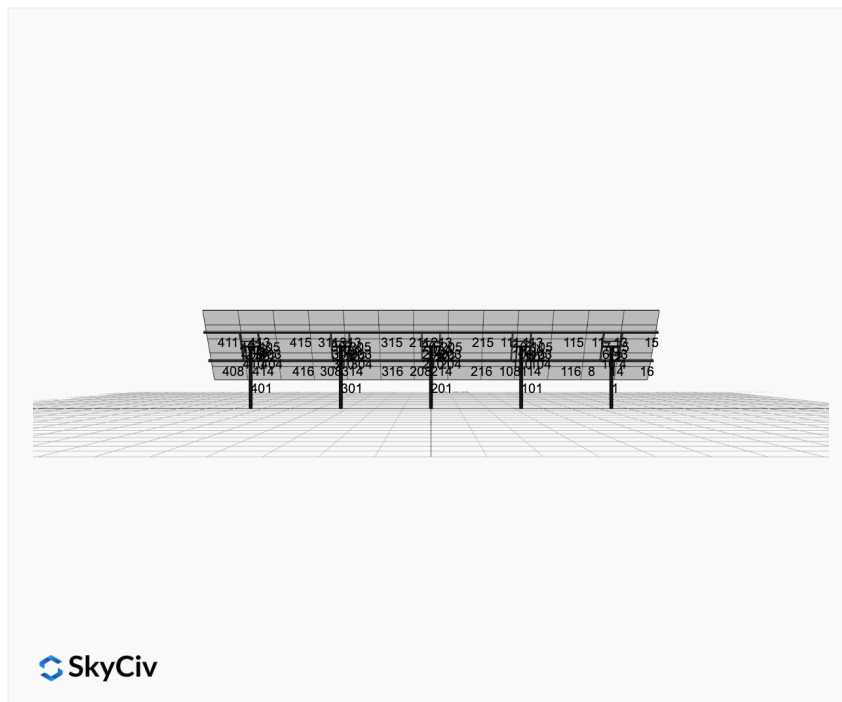
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

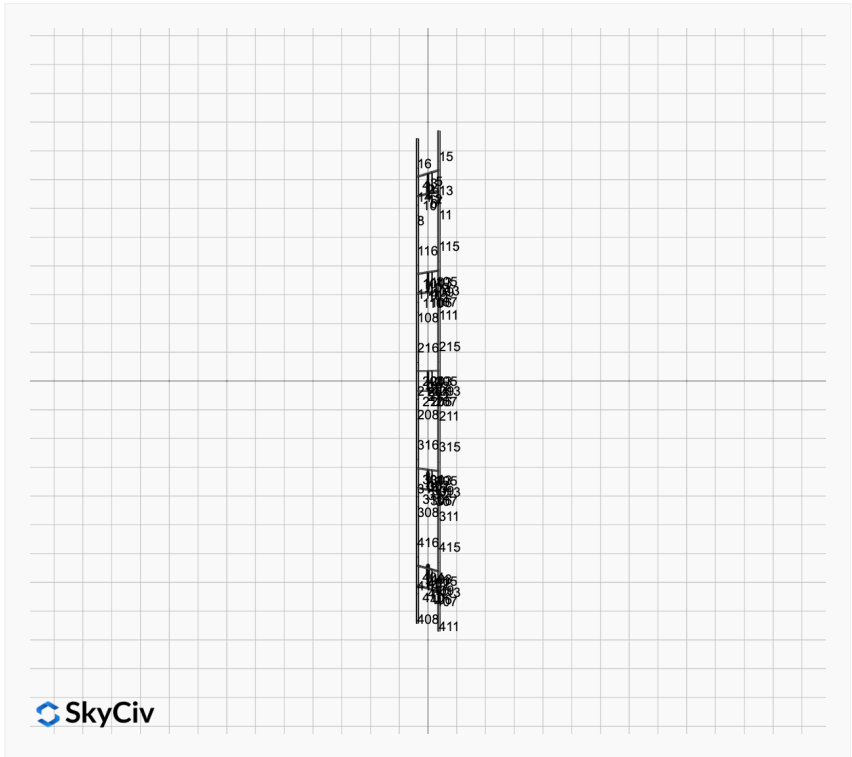
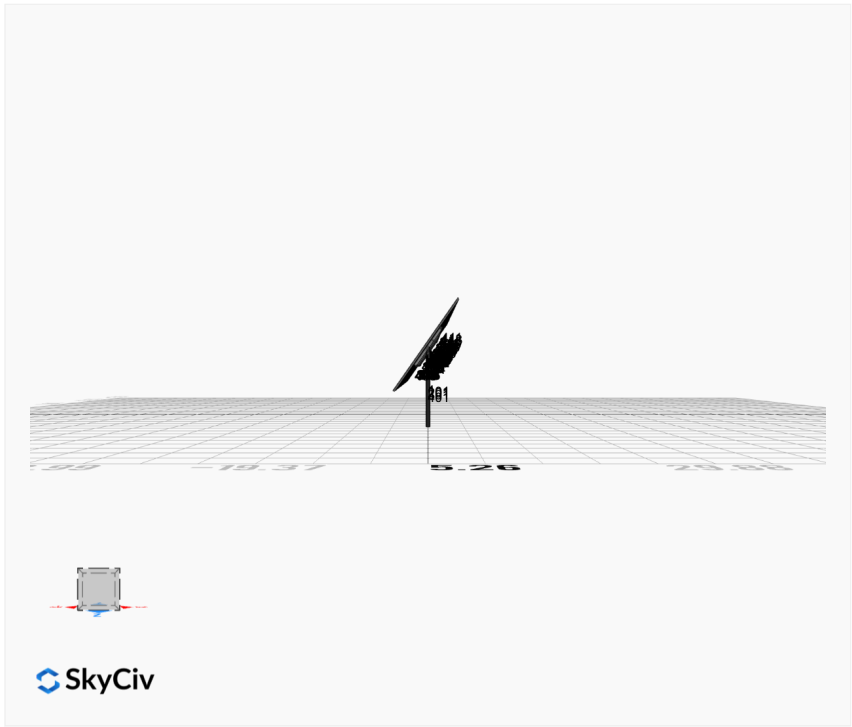
AutoDesigner Input

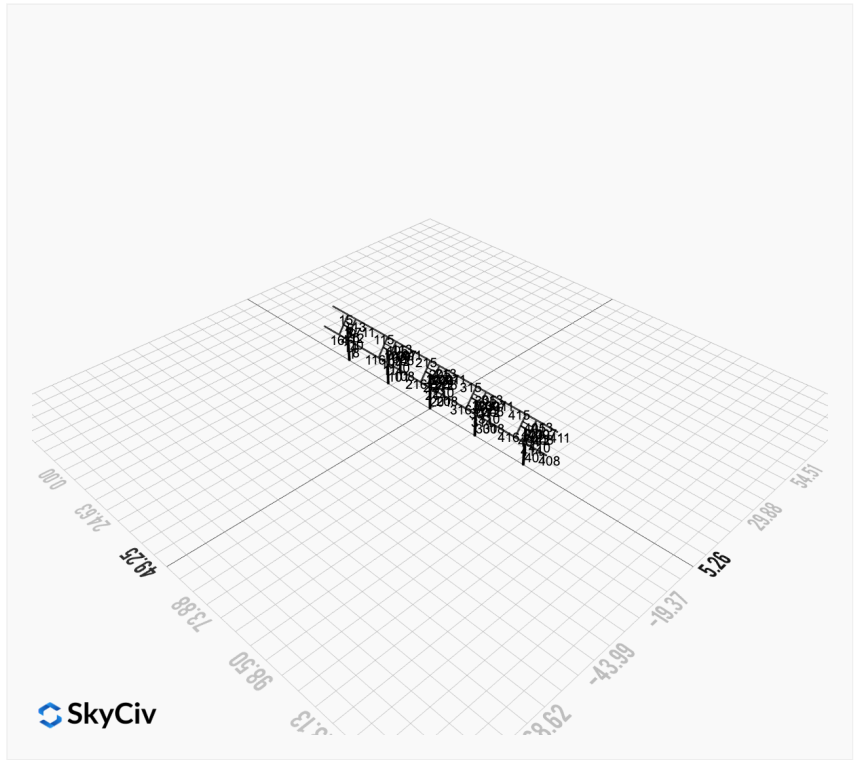
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Design Notes:

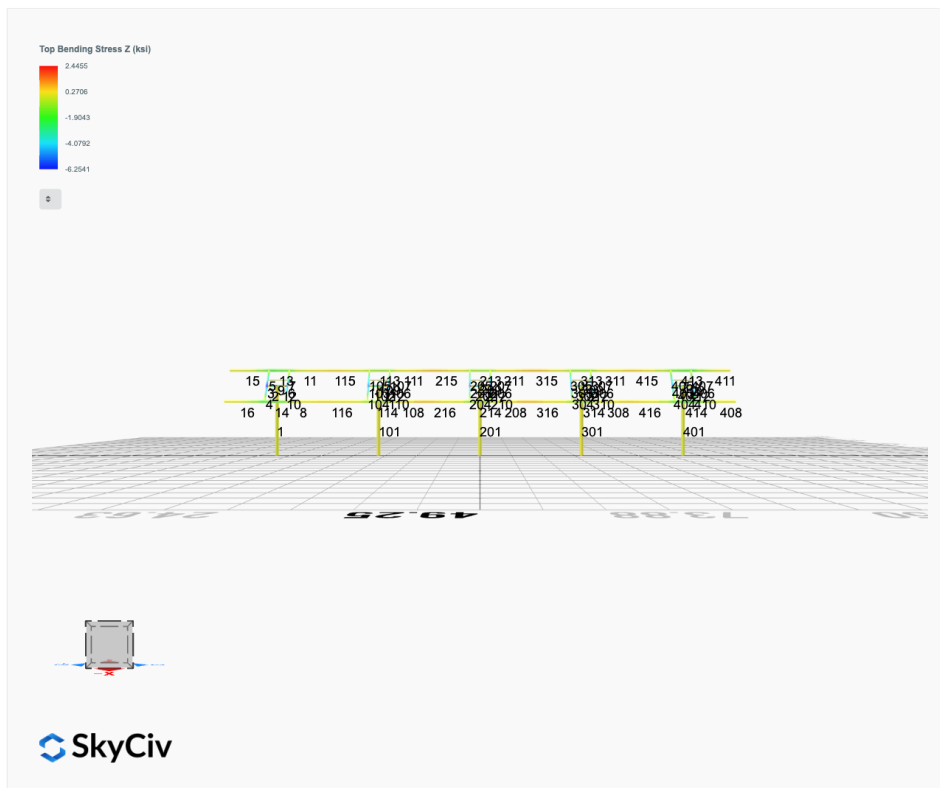
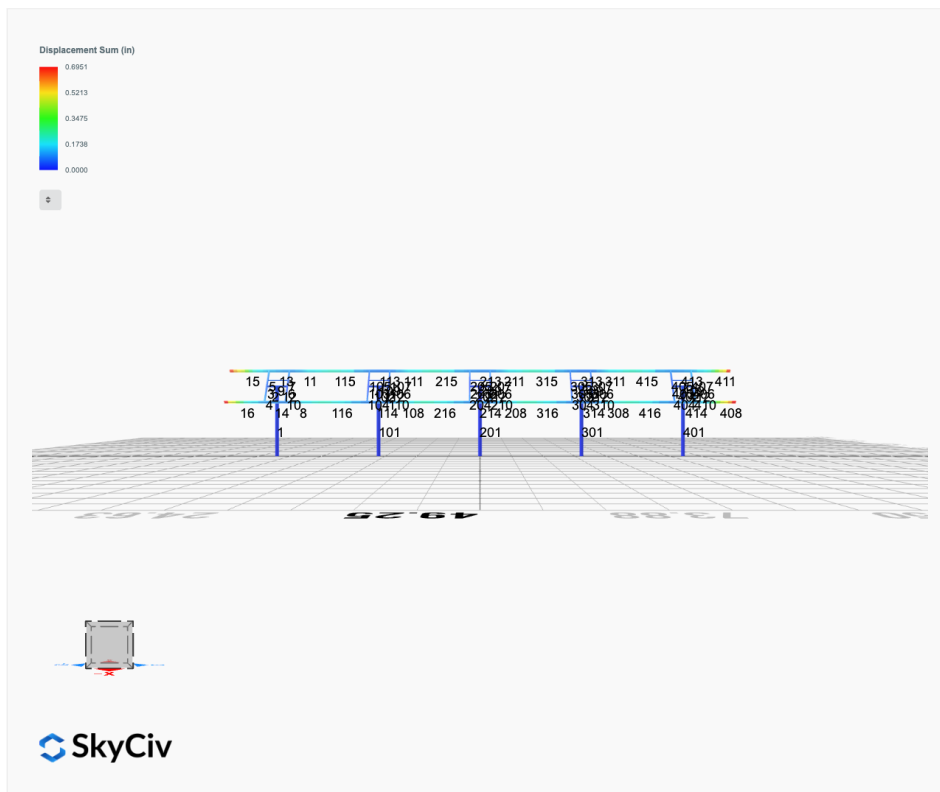
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

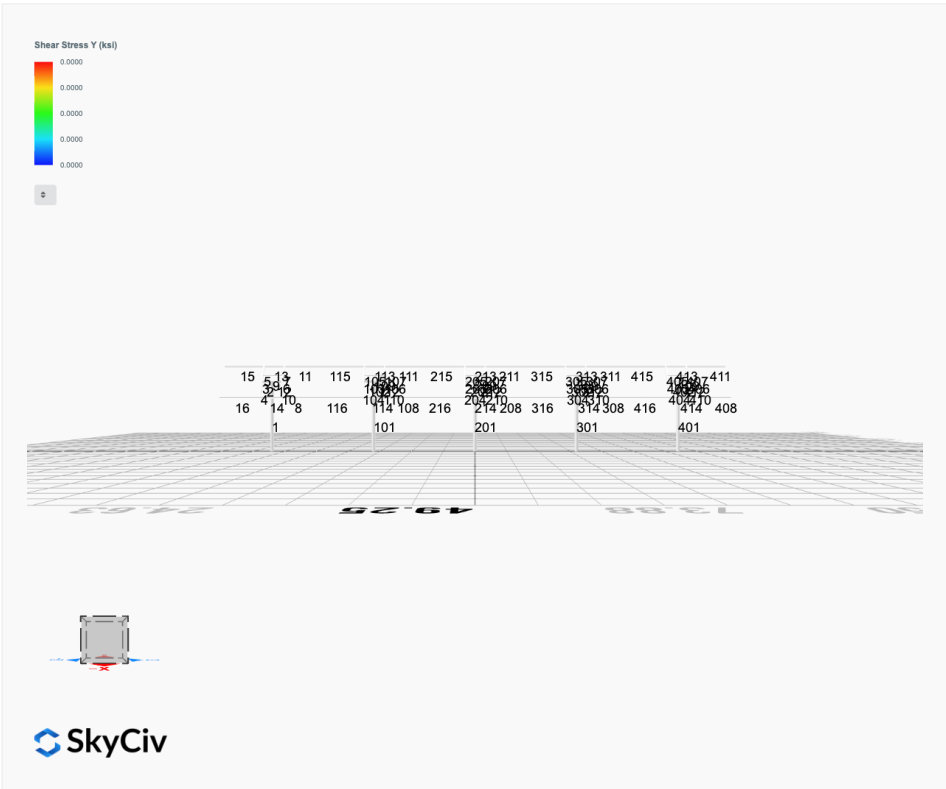
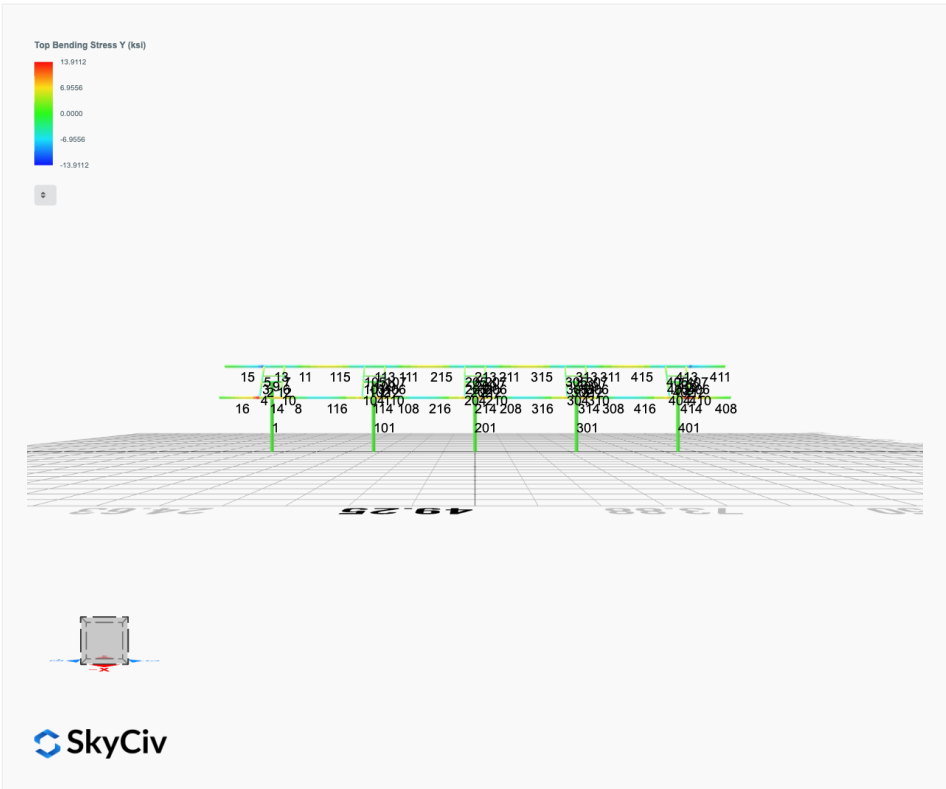






FEM Results (Envelope Worst Case for each member)





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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 2. D + L	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 3. D + (S or Lr or R)	-0.0392	4.6051	-0.0826	-0.3382	0.3025	0.5342
ULS: 3. D + (S or Lr or R)	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0341	4.0845	-0.0720	-0.2949	0.2638	0.4671
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 5b. D + 0.7E	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0341	4.0845	-0.0720	-0.2949	0.2638	0.4671
ULS: 8. 0.6D + 0.7E	-0.0115	1.5137	-0.0242	-0.0991	0.0887	0.1594
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6038	3.6363	-0.0876	-0.3536	0.4767	21.9579
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5658	1.4092	0.0068	0.0223	-0.1800	-21.0934
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2226	4.9196	-0.1075	-0.4362	0.5105	16.7362
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0341	4.0845	-0.0720	-0.2949	0.2638	0.4671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1545	3.2493	-0.0367	-0.1543	0.0179	-15.5523
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0341	4.0845	-0.0720	-0.2949	0.2638	0.4671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2076	3.3579	-0.0758	-0.3065	0.3945	16.5348
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1695	1.6876	-0.0050	-0.0246	-0.0981	-15.7536
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0191	2.5228	-0.0403	-0.1652	0.1478	0.2657
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5961	2.6272	-0.0715	-0.2875	0.4176	21.8516
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0115	1.5137	-0.0242	-0.0991	0.0887	0.1594
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5734	0.4001	0.0229	0.0883	-0.2392	-21.1997
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0115	1.5137	-0.0242	-0.0991	0.0887	0.1594

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	7.2869
Shear X	-2.6737
Shear Z	-0.1559
Moment X	-0.6355
Moment Y (Twist)	0.8031
Moment Z	37.2963

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	4.9196
Shear X	-1.6038
Shear Z	-0.1075
Moment X	-0.4362
Moment Y (Twist)	0.5105
Moment Z	21.9579

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	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 2. D + L	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 3. D + (S or Lr or R)	0.0380	4.5644	0.0073	0.0323	-0.0589	-0.4499
ULS: 3. D + (S or Lr or R)	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0331	4.0490	0.0064	0.0282	-0.0514	-0.3911
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 5b. D + 0.7E	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0331	4.0490	0.0064	0.0282	-0.0514	-0.3911
ULS: 8. 0.6D + 0.7E	0.0111	1.5017	0.0022	0.0095	-0.0173	-0.1289
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5081	3.5658	0.0115	0.0485	-0.0767	20.7326
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5448	1.4401	-0.0044	-0.0171	0.0197	-20.8441
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1119	4.8462	0.0124	0.0527	-0.0873	15.3194
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0331	4.0490	0.0064	0.0282	-0.0514	-0.3911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1778	3.2519	0.0004	0.0035	-0.0150	-15.8631
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0331	4.0490	0.0064	0.0282	-0.0514	-0.3911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1264	3.3001	0.0095	0.0403	-0.0647	15.4958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1632	1.7058	-0.0024	-0.0089	0.0076	-15.6868
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0186	2.5029	0.0036	0.0158	-0.0288	-0.2148
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5155	2.5647	0.0101	0.0422	-0.0652	20.8185
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0111	1.5017	0.0022	0.0095	-0.0173	-0.1289
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5374	0.4389	-0.0058	-0.0234	0.0312	-20.7582
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0111	1.5017	0.0022	0.0095	-0.0173	-0.1289

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	7.1879
Shear X	-2.5760
Shear Z	0.0193
Moment X	0.0817
Moment Y (Twist)	0.1286
Moment Z	35.3779

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	4.8462
Shear X	-1.5448
Shear Z	0.0124
Moment X	0.0527
Moment Y (Twist)	0.0873
Moment Z	20.8441

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	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 2. D + L	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 3. D + (S or Lr or R)	0.0024	4.6001	-0.0000	0.0000	-0.0000	-0.0475
ULS: 3. D + (S or Lr or R)	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0021	4.0801	-0.0000	0.0000	-0.0000	-0.0402
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 5b. D + 0.7E	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0021	4.0801	-0.0000	0.0000	-0.0000	-0.0402
ULS: 8. 0.6D + 0.7E	0.0007	1.5122	-0.0000	0.0000	-0.0000	-0.0109
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5565	3.6152	-0.0000	0.0000	-0.0000	21.2885
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5592	1.4253	-0.0000	0.0000	-0.0000	-21.0051
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1662	4.9013	-0.0000	0.0000	-0.0000	15.9398
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0021	4.0801	-0.0000	0.0000	-0.0000	-0.0402
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1706	3.2588	-0.0000	0.0000	-0.0000	-15.7804
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0021	4.0801	-0.0000	0.0000	-0.0000	-0.0402

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	-1.1671	3.3415	-0.0000	0.0000	-0.0000	15.9618
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1697	1.6990	-0.0000	0.0000	-0.0000	-15.7584
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0012	2.5203	-0.0000	0.0000	-0.0000	-0.0181
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5570	2.6071	-0.0000	0.0000	-0.0000	21.2957
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0007	1.5122	-0.0000	0.0000	-0.0000	-0.0109
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5587	0.4171	-0.0000	0.0000	-0.0000	-20.9979
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0007	1.5122	-0.0000	0.0000	-0.0000	-0.0109

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	7.2641
Shear X	-2.5984
Shear Z	-0.0000
Moment X	0.0001
Moment Y (Twist)	0.0001
Moment Z	36.1394

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	4.9013
Shear X	-1.5592
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	21.2957

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	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 2. D + L	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 3. D + (S or Lr or R)	0.0380	4.5644	-0.0073	-0.0323	0.0589	-0.4499
ULS: 3. D + (S or Lr or R)	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0331	4.0490	-0.0064	-0.0282	0.0513	-0.3911
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 5b. D + 0.7E	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0331	4.0490	-0.0064	-0.0282	0.0513	-0.3911
ULS: 8. 0.6D + 0.7E	0.0111	1.5017	-0.0022	-0.0095	0.0173	-0.1289
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.5081	3.5658	-0.0115	-0.0485	0.0767	20.7326
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5448	1.4401	0.0044	0.0171	-0.0197	-20.8441
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1119	4.8462	-0.0124	-0.0527	0.0873	15.3194
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0331	4.0490	-0.0064	-0.0282	0.0513	-0.3911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1778	3.2519	-0.0004	-0.0035	0.0150	-15.8631
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0331	4.0490	-0.0064	-0.0282	0.0513	-0.3911
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.1264	3.3001	-0.0095	-0.0403	0.0647	15.4958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1632	1.7058	0.0024	0.0089	-0.0076	-15.6868
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0186	2.5029	-0.0036	-0.0158	0.0288	-0.2148
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5155	2.5647	-0.0101	-0.0422	0.0652	20.8185
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0111	1.5017	-0.0022	-0.0095	0.0173	-0.1289
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5374	0.4389	0.0058	0.0234	-0.0312	-20.7582
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0111	1.5017	-0.0022	-0.0095	0.0173	-0.1289

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	7.1879
Shear X	-2.5760
Shear Z	-0.0193
Moment X	-0.0815
Moment Y (Twist)	0.1286
Moment Z	35.3780

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	4.8462
Shear X	-1.5448
Shear Z	-0.0124
Moment X	-0.0527
Moment Y (Twist)	0.0873
Moment Z	20.8441

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 2. D + L	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 3. D + (S or Lr or R)	-0.0391	4.6051	0.0826	0.3382	-0.3025	0.5342
ULS: 3. D + (S or Lr or R)	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0341	4.0845	0.0720	0.2950	-0.2638	0.4671
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 5b. D + 0.7E	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0341	4.0845	0.0720	0.2950	-0.2638	0.4671
ULS: 8. 0.6D + 0.7E	-0.0115	1.5137	0.0242	0.0991	-0.0887	0.1594
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.6038	3.6363	0.0876	0.3536	-0.4767	21.9579
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.5658	1.4092	-0.0068	-0.0222	0.1800	-21.0934
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2226	4.9196	0.1075	0.4363	-0.5105	16.7362
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0341	4.0845	0.0720	0.2950	-0.2638	0.4671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1545	3.2493	0.0367	0.1544	-0.0179	-15.5523
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0341	4.0845	0.0720	0.2950	-0.2638	0.4671
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.2076	3.3579	0.0758	0.3065	-0.3945	16.5348
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.1695	1.6876	0.0050	0.0246	0.0981	-15.7536
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0191	2.5228	0.0403	0.1652	-0.1478	0.2657
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.5961	2.6272	0.0715	0.2875	-0.4176	21.8516
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0115	1.5137	0.0242	0.0991	-0.0887	0.1594
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.5734	0.4001	-0.0229	-0.0883	0.2392	-21.1997
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0115	1.5137	0.0242	0.0991	-0.0887	0.1594

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	7.2869
Shear X	-2.6737
Shear Z	0.1559
Moment X	0.6359
Moment Y (Twist)	0.8032
Moment Z	37.2966

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	4.9196
Shear X	-1.6038
Shear Z	0.1075
Moment X	0.4363
Moment Y (Twist)	0.5105
Moment Z	21.9579

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)					
1	2in Pipe Sch 40	2.38	0.15					
4	4in Pipe Sch 40	4.50	0.24					
9	8in Pipe Sch 40	8.63	0.32					
ID	Name	d (in)	b (in)	t _w (in)	t _b (in)	r (in)		
15	HSS5x3x1/8	5.00	3.00	0.12	0.12	0.12		
ID	Name	d (in)	t _w (in)	b _t (in)	b _b (in)	t _t (in)	t _b (in)	r (in)
18	W6x9	5.90	0.17	3.94	3.94	0.21	0.21	0.25

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)
1	2in Pipe Sch 40	1.07	1.33	0.67	0.67	0.00	0.76	0.76
4	4in Pipe Sch 40	3.17	14.47	7.23	7.23	0.00	4.31	4.31
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21

412	4	1.30	1.30	2.00	-	0	0	1
413	18	4.88	4.00	7.50	1.11,1.11,1.11,1.12,1.11,1.11,1.11,1.11,1.10,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.12,1.12,1.11,1.11,1.09,1.11,1.11,1.11,1.11	300	200	1
414	18	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.08,1.10,1.10,1.10,1.10,1.10	300	200	1
415	18	6.63	6.63	10.20	1.26,1.26,1.26,1.26,1.26,1.26,1.26,1.26,1.28,1.26,1.26,1.26,1.27,1.26,1.26,1.26,1.25,1.26,1.26,1.26,1.49,1.26,1.27,1.26,1.27,1.26	300	200	1
416	18	6.63	6.63	10.20	1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.27,1.29,1.27,1.27,1.27,1.28,1.27,1.27,1.27,1.26,1.27,1.27,1.27,1.49,1.27,1.27,1.27,1.28,1.27	300	200	1

Member Design Capacity

Member ID	$\Phi_c P_n$ (kip)	$\Phi_t P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	139.54	83.29	83.29	113.39	113.39
2	142.83	141.72	16.17	16.17	42.85	42.85
3	79.65	74.02	10.99	4.60	29.14	16.61
4	79.65	72.01	10.99	4.60	29.14	16.61
5	79.65	73.44	10.99	4.60	29.14	16.61
6	79.65	74.02	10.99	4.60	29.14	16.61
7	79.65	73.44	10.99	4.60	29.14	16.61
8	120.60	117.88	23.36	6.45	30.09	45.74
9	48.35	43.11	2.85	2.85	14.51	14.51
10	79.65	72.01	10.99	4.60	29.14	16.61
11	120.60	117.88	23.36	6.45	30.09	45.74
12	142.83	141.72	16.17	16.17	42.85	42.85
13	120.60	98.23	19.19	6.45	30.09	45.74
14	120.60	98.23	19.01	6.45	30.09	45.74
15	120.60	21.74	23.36	6.45	30.09	45.74
16	120.60	21.74	23.36	6.45	30.09	45.74
101	377.97	139.54	83.29	83.29	113.39	113.39
102	142.83	141.72	16.17	16.17	42.85	42.85
103	79.65	74.02	10.99	4.60	29.14	16.61
104	79.65	72.01	10.99	4.60	29.14	16.61
105	79.65	73.44	10.99	4.60	29.14	16.61
106	79.65	74.02	10.99	4.60	29.14	16.61
107	79.65	73.44	10.99	4.60	29.14	16.61
108	120.60	117.88	23.36	6.45	30.09	45.74
109	48.35	43.11	2.85	2.85	14.51	14.51
110	79.65	72.01	10.99	4.60	29.14	16.61
111	120.60	117.88	23.36	6.45	30.09	45.74
112	142.83	141.72	16.17	16.17	42.85	42.85
113	120.60	98.23	18.49	6.45	30.09	45.74
114	120.60	98.23	18.31	6.45	30.09	45.74
115	120.60	68.63	15.85	6.45	30.09	45.74
116	120.60	68.63	15.85	6.45	30.09	45.74
201	377.97	139.54	83.29	83.29	113.39	113.39
202	142.83	141.72	16.17	16.17	42.85	42.85
203	79.65	74.02	10.99	4.60	29.14	16.61
204	79.65	72.01	10.99	4.60	29.14	16.61
205	79.65	73.44	10.99	4.60	29.14	16.61
206	79.65	74.02	10.99	4.60	29.14	16.61
207	79.65	73.44	10.99	4.60	29.14	16.61

208	120.60	117.88	23.36	6.45	30.09	45.74
209	48.35	43.11	2.85	2.85	14.51	14.51
210	79.65	72.01	10.99	4.60	29.14	16.61
211	120.60	117.88	23.36	6.45	30.09	45.74
212	142.83	141.72	16.17	16.17	42.85	42.85
213	120.60	98.23	18.31	6.45	30.09	45.74
214	120.60	98.23	18.31	6.45	30.09	45.74
215	120.60	68.63	15.57	6.45	30.09	45.74
216	120.60	68.63	15.71	6.45	30.09	45.74
301	377.97	139.54	83.29	83.29	113.39	113.39
302	142.83	141.72	16.17	16.17	42.85	42.85
303	79.65	74.02	10.99	4.60	29.14	16.61
304	79.65	72.01	10.99	4.60	29.14	16.61
305	79.65	73.44	10.99	4.60	29.14	16.61
306	79.65	74.02	10.99	4.60	29.14	16.61
307	79.65	73.44	10.99	4.60	29.14	16.61
308	120.60	117.88	23.36	6.45	30.09	45.74
309	48.35	43.11	2.85	2.85	14.51	14.51
310	79.65	72.01	10.99	4.60	29.14	16.61
311	120.60	117.88	23.36	6.45	30.09	45.74
312	142.83	141.72	16.17	16.17	42.85	42.85
313	120.60	98.23	18.31	6.45	30.09	45.74
314	120.60	98.23	18.49	6.45	30.09	45.74
315	120.60	68.63	15.71	6.45	30.09	45.74
316	120.60	68.63	15.57	6.45	30.09	45.74
401	377.97	139.54	83.29	83.29	113.39	113.39
402	142.83	141.72	16.17	16.17	42.85	42.85
403	79.65	74.02	10.99	4.60	29.14	16.61
404	79.65	72.01	10.99	4.60	29.14	16.61
405	79.65	73.44	10.99	4.60	29.14	16.61
406	79.65	74.02	10.99	4.60	29.14	16.61
407	79.65	73.44	10.99	4.60	29.14	16.61
408	120.60	21.74	23.36	6.45	30.09	45.74
409	48.35	43.11	2.85	2.85	14.51	14.51
410	79.65	72.01	10.99	4.60	29.14	16.61
411	120.60	21.74	23.36	6.45	30.09	45.74
412	142.83	141.72	16.17	16.17	42.85	42.85
413	120.60	98.23	19.19	6.45	30.09	45.74
414	120.60	98.23	19.01	6.45	30.09	45.74
415	120.60	68.63	17.08	6.45	30.09	45.74
416	120.60	68.63	17.21	6.45	30.09	45.74

Design Ratio

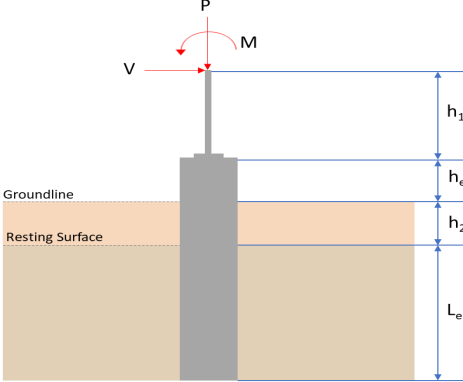
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.052	0.448	0.018	0.024	0.001	0.476	#13	0.583	Not Required	Pass
2	0.006	0.356	0.212	0.089	0.034	0.521	#13	0.034	Not Required	Pass
3	0.018	0.499	0.166	0.051	0.013	0.649	#21	0.044	Not Required	Pass
4	0.017	0.496	0.512	0.050	0.061	0.771	#21	0.078	Not Required	Pass
5	0.018	0.309	0.544	0.050	0.079	0.553	#23	0.073	Not Required	Pass
6	0.015	0.406	0.115	0.040	0.005	0.481	#21	0.044	Not Required	Pass
7	0.014	0.253	0.407	0.040	0.062	0.414	#21	0.073	Not Required	Pass
8	0.002	0.058	0.137	0.029	0.016	0.192	#21	0.088	Not Required	Pass

9	0.026	0.031	0.075	0.002	0.002	0.109	#13	0.198	Not Required	Pass
10	0.013	0.408	0.407	0.041	0.051	0.672	#21	0.078	Not Required	Pass
11	0.002	0.055	0.139	0.028	0.016	0.192	#21	0.059	Not Required	Pass
12	0.006	0.265	0.163	0.070	0.029	0.385	#13	0.034	Not Required	Pass
13	0.011	0.200	0.551	0.036	0.020	0.740	#21	0.177	Not Required	Pass
14	0.010	0.204	0.551	0.036	0.020	0.742	#21	0.177	Not Required	Pass
15	0.000	0.100	0.330	0.026	0.016	0.425	#21	Not Required	Not Required	Pass
16	0.000	0.100	0.330	0.026	0.016	0.425	#21	Not Required	Not Required	Pass
101	0.052	0.425	0.002	0.023	0.000	0.444	#13	0.583	Not Required	Pass
102	0.003	0.299	0.167	0.078	0.030	0.417	#13	0.034	Not Required	Pass
103	0.016	0.433	0.158	0.043	0.017	0.576	#21	0.044	Not Required	Pass
104	0.015	0.424	0.352	0.043	0.044	0.655	#21	0.078	Not Required	Pass
105	0.016	0.269	0.363	0.043	0.053	0.371	#23	0.073	Not Required	Pass
106	0.016	0.445	0.161	0.045	0.017	0.591	#21	0.044	Not Required	Pass
107	0.016	0.276	0.366	0.044	0.053	0.374	#23	0.073	Not Required	Pass
108	0.002	0.039	0.134	0.026	0.016	0.151	#21	0.088	Not Required	Pass
109	0.014	0.023	0.046	0.001	0.000	0.072	#13	0.198	Not Required	Pass
110	0.015	0.439	0.353	0.044	0.044	0.664	#21	0.078	Not Required	Pass
111	0.002	0.035	0.135	0.027	0.016	0.157	#21	0.059	Not Required	Pass
112	0.003	0.309	0.175	0.079	0.030	0.436	#13	0.034	Not Required	Pass
113	0.010	0.126	0.359	0.034	0.020	0.469	#21	0.177	Not Required	Pass
114	0.010	0.122	0.359	0.034	0.020	0.462	#21	0.265	Not Required	Pass
115	0.002	0.091	0.207	0.025	0.016	0.295	#21	0.293	Not Required	Pass
116	0.003	0.094	0.212	0.024	0.016	0.300	#21	0.439	Not Required	Pass
201	0.052	0.434	0.000	0.023	0.000	0.455	#13	0.583	Not Required	Pass
202	0.003	0.307	0.175	0.079	0.030	0.435	#13	0.034	Not Required	Pass
203	0.016	0.445	0.158	0.045	0.017	0.588	#21	0.044	Not Required	Pass
204	0.015	0.446	0.356	0.045	0.045	0.675	#21	0.078	Not Required	Pass
205	0.016	0.276	0.369	0.044	0.054	0.377	#21	0.073	Not Required	Pass
206	0.016	0.445	0.158	0.045	0.017	0.588	#21	0.044	Not Required	Pass
207	0.016	0.276	0.369	0.044	0.054	0.377	#21	0.073	Not Required	Pass
208	0.002	0.037	0.136	0.027	0.016	0.157	#21	0.088	Not Required	Pass
209	0.014	0.022	0.045	0.001	0.000	0.070	#13	0.198	Not Required	Pass
210	0.015	0.446	0.356	0.045	0.045	0.675	#21	0.078	Not Required	Pass
211	0.002	0.038	0.137	0.026	0.016	0.157	#21	0.059	Not Required	Pass
212	0.003	0.307	0.175	0.079	0.030	0.435	#13	0.034	Not Required	Pass
213	0.010	0.121	0.362	0.034	0.020	0.469	#21	0.177	Not Required	Pass
214	0.010	0.127	0.361	0.034	0.020	0.471	#21	0.265	Not Required	Pass
215	0.002	0.123	0.207	0.026	0.016	0.324	#21	0.293	Not Required	Pass
216	0.003	0.125	0.211	0.027	0.016	0.329	#21	0.439	Not Required	Pass
301	0.052	0.425	0.002	0.023	0.000	0.444	#13	0.583	Not Required	Pass
302	0.003	0.309	0.175	0.079	0.030	0.436	#13	0.034	Not Required	Pass
303	0.016	0.445	0.161	0.045	0.017	0.591	#21	0.044	Not Required	Pass
304	0.015	0.439	0.353	0.044	0.044	0.664	#21	0.078	Not Required	Pass
305	0.016	0.276	0.366	0.044	0.053	0.374	#23	0.073	Not Required	Pass
306	0.016	0.433	0.158	0.043	0.017	0.576	#21	0.044	Not Required	Pass
307	0.016	0.269	0.363	0.043	0.053	0.371	#23	0.073	Not Required	Pass
308	0.002	0.029	0.132	0.024	0.016	0.152	#21	0.088	Not Required	Pass
309	0.014	0.023	0.046	0.001	0.000	0.072	#13	0.198	Not Required	Pass
310	0.015	0.424	0.352	0.043	0.044	0.655	#21	0.078	Not Required	Pass
311	0.002	0.029	0.133	0.025	0.016	0.158	#21	0.059	Not Required	Pass
312	0.003	0.299	0.167	0.078	0.030	0.417	#13	0.034	Not Required	Pass
313	0.010	0.126	0.359	0.034	0.020	0.469	#21	0.177	Not Required	Pass
314	0.010	0.122	0.359	0.034	0.020	0.462	#21	0.265	Not Required	Pass

315	0.002	0.122	0.207	0.027	0.016	0.323	#21	0.293	Not Required	Pass
316	0.003	0.124	0.211	0.026	0.016	0.329	#21	0.439	Not Required	Pass
401	0.052	0.448	0.018	0.024	0.001	0.476	#13	0.583	Not Required	Pass
402	0.006	0.265	0.163	0.070	0.029	0.385	#13	0.034	Not Required	Pass
403	0.015	0.406	0.115	0.040	0.005	0.481	#21	0.044	Not Required	Pass
404	0.013	0.408	0.407	0.041	0.051	0.672	#21	0.078	Not Required	Pass
405	0.014	0.253	0.407	0.040	0.062	0.414	#21	0.073	Not Required	Pass
406	0.018	0.499	0.166	0.051	0.013	0.649	#21	0.044	Not Required	Pass
407	0.018	0.309	0.544	0.049	0.079	0.553	#23	0.073	Not Required	Pass
408	0.000	0.100	0.330	0.026	0.016	0.425	#21	Not Required	Not Required	Pass
409	0.026	0.031	0.075	0.002	0.002	0.109	#13	0.198	Not Required	Pass
410	0.017	0.496	0.512	0.050	0.061	0.771	#21	0.078	Not Required	Pass
411	0.000	0.100	0.330	0.026	0.016	0.425	#21	Not Required	Not Required	Pass
412	0.006	0.356	0.212	0.089	0.034	0.521	#13	0.034	Not Required	Pass
413	0.011	0.200	0.551	0.036	0.020	0.740	#21	0.177	Not Required	Pass
414	0.010	0.204	0.551	0.036	0.020	0.742	#21	0.265	Not Required	Pass
415	0.002	0.085	0.208	0.028	0.016	0.289	#21	0.293	Not Required	Pass
416	0.003	0.086	0.212	0.029	0.016	0.294	#21	0.439	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 $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="512 1102 1096 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>Sedimentary & foliated rock</td> <td>4000.000</td> <td>400.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.920</td> <td>7.287</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.604</td> <td>-2.674</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.107</td> <td>-0.156</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.436</td> <td>-0.636</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.958</td> <td>37.296</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	Sedimentary & foliated rock	4000.000	400.000	Load Component	ASD	LRFD	P (kip)	4.920	7.287	V_x (kip)	-1.604	-2.674	V_z (kip)	-0.107	-0.156	M_x (kipft)	-0.436	-0.636	M_z (kipft)	21.958	37.296	
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M_z (kipft)	21.958	37.296																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.604 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25541 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.958 \text{ kipft}) + ((-1.604 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4965 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.311 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.107 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.017038 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.436 \text{ kipft}) + ((-0.107 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.069427 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1772 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.311 \text{ ft}), (1.1772 \text{ ft})]$$

$$L_{e,req} = 4.311 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.311 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.958$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.92 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.3075 \text{ kip/ft}^2$$

$$q = 0.3075 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.3075 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.076875$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25541 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.4965 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4965 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (3.4965 \text{ kipft/ft})) + (4 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0674 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.4965 \text{ kipft/ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (3.4965 \text{ kipft/ft})) + (2 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.50213 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (3.4965 \text{ kipft/ft})) + ((-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 1.7314 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.0674 \text{ ft})}{2}$$

$$p_a = 0.61348 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.50213 \text{ kip/ft}^2)}{(0.61348 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8185$$

p_a - Allowable lateral soil pressure at depth L_e ,

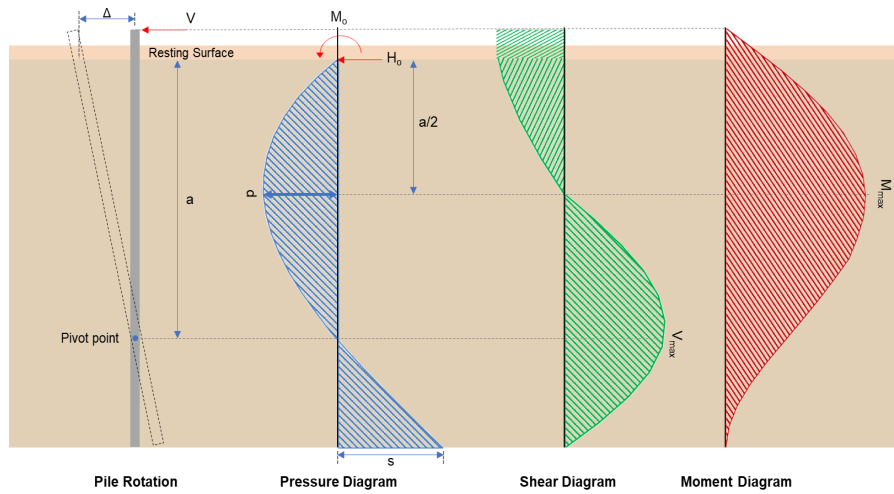
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.7314 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.96191$	<p>Status: PASS Ratio: 0.960</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.017038 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.069427 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.069427 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.017038 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.069427 \text{ kipft/ft})) + (4 \times (-0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))}$ $a = 3.159 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.069427 \text{ kipft/ft})) + (3 \times (-0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (0.069427 \text{ kipft/ft})) + (2 \times (-0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$ $p = 0.0015334 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.069427 \text{ kipft/ft})) + ((-0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$ $s = 0.018424 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (400 \text{ psf/ft}) \times \frac{(3.159 \text{ ft})}{2}$ $p_a = 0.6318 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0015334 \text{ kip/ft}^2)}{(0.6318 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.002427$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.000</p>

$$Ratio = \frac{(0.018424 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$Ratio = 0.010236$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.674 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4258 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(37.296 \text{ kipft}) + ((-2.674 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.9389 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.9389 \text{ kipft/ft})}{(-0.4258 \text{ kip/ft})}$$

$$E = 13.948 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.9389 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.4258 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (5.9389 \text{ kipft/ft})) + (4 \times (-0.4258 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0664 \text{ ft}$$

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

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

$$V_{max} = ((-0.4258 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0664 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0664 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.474 \text{ kip}$$

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

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

$$M_{max} = ((-0.4258 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(13.948 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.866 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.156 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.024841 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.636 \text{ kipft}) + ((-0.156 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10127 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10127 \text{ kipft/ft})}{(-0.024841 \text{ kip/ft})}$$

$$E = 4.0769 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10127 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.024841 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.10127 \text{ kipft/ft})) + (4 \times (-0.024841 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.159 \text{ ft}$$

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

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

$$V_{max} = ((-0.024841 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.159 \text{ ft})}{(4.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.159 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.22498 \text{ kip}$$

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

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

$$M_{max} = ((-0.024841 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.0769 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.46598 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.287 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.354 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.354 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(7.287 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0027239$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.287 \text{ kip} \rightarrow 7287 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7287 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.46 \text{ kip}$$

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

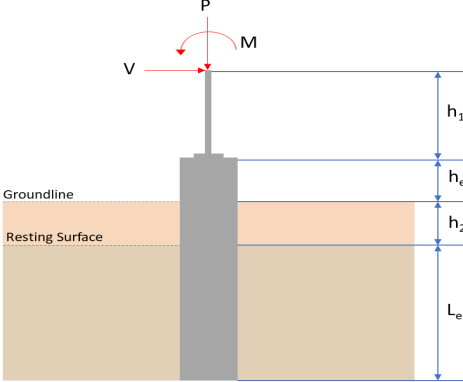
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.46 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.46 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.46 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.73 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.474 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.474 \text{ kip})}{(110.73 \text{ kip})}$ $\text{Ratio} = 0.094593$ <p>Considering z-direction:</p> <p>$V_{max} = 0.22498 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.22498 \text{ kip})}{(110.73 \text{ kip})}$ $\text{Ratio} = 0.0020318$	<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> $S_m = \frac{b D^3}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<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: $M_{max} = 22.866 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(22.866 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.091611$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.46598 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.46598 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0018669$	<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 $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="512 1102 1096 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>Sedimentary & foliated rock</td> <td>4000.000</td> <td>400.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.846</td> <td>7.188</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.545</td> <td>-2.576</td> </tr> <tr> <td>V_z (kip)</td> <td>0.012</td> <td>0.019</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.053</td> <td>0.082</td> </tr> <tr> <td>M_z (kipft)</td> <td>20.844</td> <td>35.378</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	Sedimentary & foliated rock	4000.000	400.000	Load Component	ASD	LRFD	P (kip)	4.846	7.188	V_x (kip)	-1.545	-2.576	V_z (kip)	0.012	0.019	M_x (kipft)	0.053	0.082	M_z (kipft)	20.844	35.378	
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M_z (kipft)	20.844	35.378																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.545 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24602 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(20.844 \text{ kipft}) + ((-1.545 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3191 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.2379 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.012 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0019108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.053 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0084395 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.65527 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.2379 \text{ ft}), (0.65527 \text{ ft})]$$

$$L_{e,req} = 4.238 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.238 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.94178$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.846 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.302875 \text{ kip/ft}^2$$

$$q = 0.00200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.30288 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.075719$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.24602 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3191 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3191 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (3.3191 \text{ kipft/ft})) + (4 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0682 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.3191 \text{ kipft/ft})) + (3 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (3.3191 \text{ kipft/ft})) + (2 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.47404 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (3.3191 \text{ kipft/ft})) + ((-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 1.6389 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.0682 \text{ ft})}{2}$$

$$p_a = 0.61364 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.47404 \text{ kip/ft}^2)}{(0.61364 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7725$$

p_a - Allowable lateral soil pressure at depth L_e ,

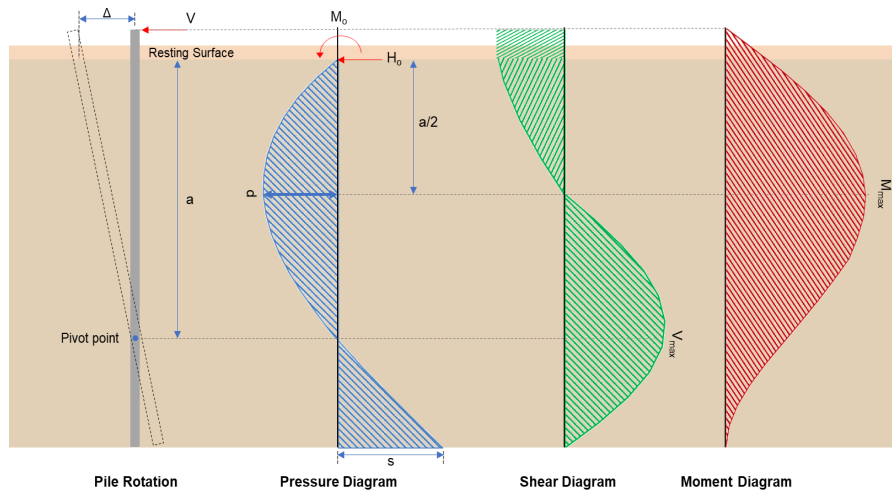
Status: **PASS**
Ratio: **0.770**

	$p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.6389 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91047$	Status: PASS Ratio: 0.910
	<p>Considering z-direction:</p> <p>$H_o = 0.0019108 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0084395 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0084395 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.0084395 \text{ kipft/ft})) + (4 \times (0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))}$ $a = 3.1517 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0084395 \text{ kipft/ft})) + (3 \times (0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.0084395 \text{ kipft/ft})) + (2 \times (0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$ $p = 0.0030896 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0084395 \text{ kipft/ft})) + ((0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$ $s = 0.007549 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (400 \text{ psf/ft}) \times \frac{(3.1517 \text{ ft})}{2}$ $p_a = 0.63034 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0030896 \text{ kip/ft}^2)}{(0.63034 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0049016$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.000

$$\text{Ratio} = \frac{(0.007549 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0041939$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.576 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.41019 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.378 \text{ kipft}) + ((-2.576 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6334 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6334 \text{ kipft/ft})}{(-0.41019 \text{ kip/ft})}$$

$$E = 13.734 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6334 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.41019 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (5.6334 \text{ kipft/ft})) + (4 \times (-0.41019 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0672 \text{ ft}$$

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

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

$$V_{max} = ((-0.41019 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0672 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0672 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.9518 \text{ kip}$$

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

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

$$M_{max} = ((-0.41019 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(13.734 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 21.717 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.019 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0030255 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.082 \text{ kipft}) + ((0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013057 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013057 \text{ kipft/ft})}{(0.0030255 \text{ kip/ft})}$$

$$E = 4.3158 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013057 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.0030255 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.013057 \text{ kipft/ft})) + (4 \times (0.0030255 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1538 \text{ ft}$$

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

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

$$V_{max} = ((0.0030255 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1538 \text{ ft})}{(4.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1538 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.028534 \text{ kip}$$

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

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

$$M_{max} = ((0.0030255 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.3158 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.059298 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.188 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.357 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.357 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(7.188 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0026869$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.188 \text{ kip} \rightarrow 7188 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7188 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.44 \text{ kip}$$

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

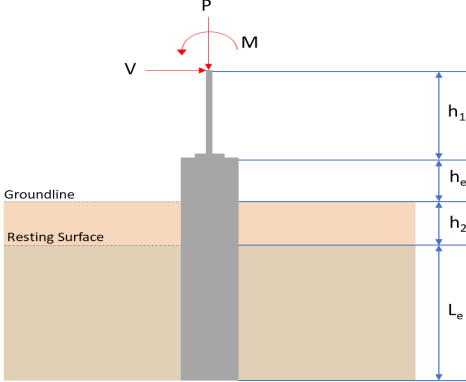
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.44 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.44 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.44 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.72 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.9518 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.9518 \text{ kip})}{(110.72 \text{ kip})}$ $\text{Ratio} = 0.089883$ <p>Considering z-direction:</p> <p>$V_{max} = 0.028534 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.028534 \text{ kip})}{(110.72 \text{ kip})}$ $\text{Ratio} = 0.00025771$	<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: $M_{max} = 21.717 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(21.717 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.087006$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.059298 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.059298 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00023757$	<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 $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="512 1102 1098 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>Sedimentary & foliated rock</td> <td>4000.000</td> <td>400.000</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>4.901</td> <td>7.264</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.559</td> <td>-2.598</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.296</td> <td>36.139</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	Sedimentary & foliated rock	4000.000	400.000	Load Component	ASD	LRFD	P (kip)	4.901	7.264	V_x (kip)	-1.559	-2.598	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	21.296	36.139	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sedimentary & foliated rock	4000.000	400.000																									
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V_z (kip)	0.000	0.000																										
M_x (kipft)	0.000	0.000																										
M_z (kipft)	21.296	36.139																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.559 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24825 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

	$M_o = \frac{(21.296 \text{ kipft}) + ((-1.559 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$ $M_o = 3.3911 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 4.2705 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(4.2705 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 4.27 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_e - h_2$ $L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 4.5 \text{ ft}$ <p><i>Ratio</i> - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(4.27 \text{ ft})}{(4.5 \text{ ft})}$ $\text{Ratio} = 0.94889$	<p>Status: PASS Ratio: 0.950</p>
	<p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = b D$ $A = (48 \text{ in}) \times (48 \text{ in})$ $A = 16 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_u}{A}$ $q = \frac{(4.901 \text{ kip})}{(16 \text{ ft}^2)}$ $q = 0.30631 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p><i>Ratio</i> - Capacity</p> $\text{Ratio} = \frac{q}{q_o}$ $\text{Ratio} = \frac{(0.30631 \text{ kip/ft}^2)}{(4000 \text{ psf})}$ $\text{Ratio} = 0.076578$	<p>Status: PASS Ratio: 0.080</p>
<p>Czerniak</p>	<p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$	

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.24825$ kip/ft - Lateral force per length of pile,

$M_o = 3.3911$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3911 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.24825 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (3.3911 \text{ kipft/ft})) + (4 \times (-0.24825 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0675 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3911 \text{ kipft/ft})) + (3 \times (-0.24825 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (3.3911 \text{ kipft/ft})) + (2 \times (-0.24825 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.4866 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3911 \text{ kipft/ft})) + ((-0.24825 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 1.6785 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.0675 \text{ ft})}{2}$$

$$p_a = 0.61351 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.4866 \text{ kip/ft}^2)}{(0.61351 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.79315$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 1.8 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

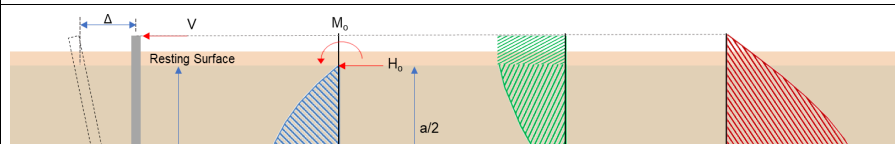
$$\text{Ratio} = \frac{s}{p_s}$$

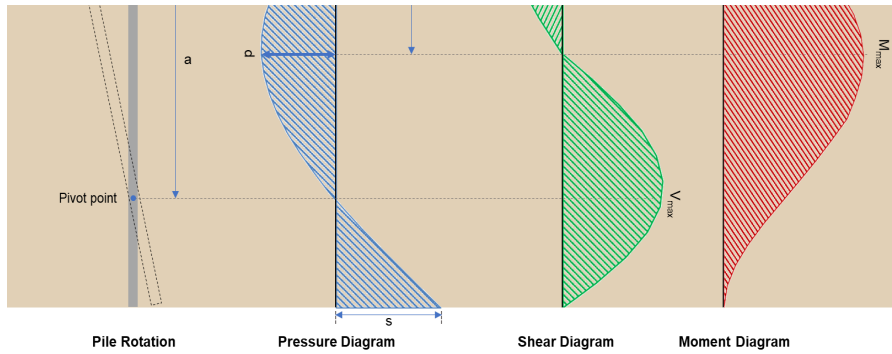
$$\text{Ratio} = \frac{(1.6785 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93252$$

Status: **PASS**
Ratio: **0.790**

Status: **PASS**
Ratio: **0.930**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.598 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.41369 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(36.139 \text{ kipft}) + ((-2.598 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.7546 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.7546 \text{ kipft/ft})}{(-0.41369 \text{ kip/ft})}$$

$$E = 13.91 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.7546 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.41369 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (5.7546 \text{ kipft/ft})) + (4 \times (-0.41369 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0665 \text{ ft}$$

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

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

$$V_{max} = ((-0.41369 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.91 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0665 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.91 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0665 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.152 \text{ kip}$$

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

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

$$M_{max} = ((-0.41369 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(13.91 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0665 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.91 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0665 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.91 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0665 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.161 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.264 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.355 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.355 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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)**

22.4.2.2	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}] + (f_{yk} A_{st}))]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p>Ratio - Capacity</p> $\text{Ratio} = \frac{P}{\phi P_N}$ $\text{Ratio} = \frac{(7.264 \text{ kip})}{(2675.2 \text{ kip})}$ $\text{Ratio} = 0.0027153$	Status: PASS Ratio: 0.000
22.5.2.2 22.5.5.1.3 22.5.5.1.1 22.5.5.1.1(a) 22.5.5.1.2	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,max} = 296.21 \text{ kip}$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.264 \text{ kip} \rightarrow 7264 \text{ lbf}$.</p> <p>$V_{c,a}$ - Shear strength of concrete (a)</p> $V_{c,a} = \left[2 \lambda_s \sqrt{f'_{ck}} + \frac{P}{6 A_g} \right] b_w d$ $V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7264 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,a} = 119.45 \text{ kip}$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{c,b}$ - Shear strength of concrete (b)</p> $V_{c,b} = \left[2 \lambda_s \sqrt{f'_{ck}} + (0.05 f'_{ck}) \right] b_w d$ $V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{c,b} = 348.89 \text{ kip}$ <p>V_c - Governing shear strength of concrete</p> $V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$ $V_c = \text{Min}[(296.21 \text{ kip}), (119.45 \text{ kip}), (348.89 \text{ kip})]$ $V_c = 119.45 \text{ kip}$	

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.45 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.73 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.152 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.152 \text{ kip})}{(110.73 \text{ kip})}$ $\text{Ratio} = 0.091686$	<p>Status: PASS Ratio: 0.090</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kip ft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = 0.85 f'_c S_m$	

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

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

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

Therefore,
 ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

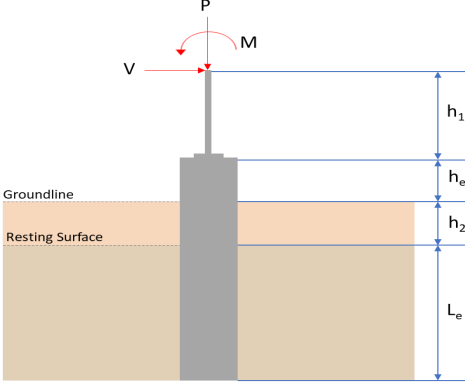
$M_{max} = 22.161 \text{ kipft}$ - Maximum moment in the x-direction,
Ratio - Capacity

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

$$\text{Ratio} = \frac{(22.161 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.088788$$

Status: **PASS**
Ratio: **0.090**

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 $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resisting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="512 1102 1098 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>Sedimentary & foliated rock</td> <td>4000.000</td> <td>400.000</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>4.846</td> <td>7.188</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.545</td> <td>-2.576</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.012</td> <td>-0.019</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.053</td> <td>-0.082</td> </tr> <tr> <td>M_z (kipft)</td> <td>20.844</td> <td>35.378</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	Sedimentary & foliated rock	4000.000	400.000	Load Component	ASD	LRFD	P (kip)	4.846	7.188	V_x (kip)	-1.545	-2.576	V_z (kip)	-0.012	-0.019	M_x (kipft)	-0.053	-0.082	M_z (kipft)	20.844	35.378	
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M_z (kipft)	20.844	35.378																										
	<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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.545 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.24602 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(20.844 \text{ kipft}) + ((-1.545 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3191 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 4.2379 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.012 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0019108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.053 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0084395 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.61 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.2379 \text{ ft}), (0.61 \text{ ft})]$$

$$L_{e,req} = 4.238 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.238 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.94178$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.846 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.302875 \text{ kip/ft}^2$$

$$q = 0.00200 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$Ratio = \frac{(0.30288 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$Ratio = 0.075719$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.24602 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3191 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3191 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (3.3191 \text{ kipft/ft})) + (4 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0682 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.3191 \text{ kipft/ft})) + (3 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (3.3191 \text{ kipft/ft})) + (2 \times (-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.47404 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (3.3191 \text{ kipft/ft})) + ((-0.24602 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 1.6389 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.0682 \text{ ft})}{2}$$

$$p_a = 0.61364 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.47404 \text{ kip/ft}^2)}{(0.61364 \text{ kip/ft}^2)}$$

$$Ratio = 0.7725$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.770**

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 1.8 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.6389 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91047$$

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = -0.0019108 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0084395 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0084395 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.0084395 \text{ kipft/ft})) + (4 \times (-0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1517 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0084395 \text{ kipft/ft})) + (3 \times (-0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 \times [(3 \times (0.0084395 \text{ kipft/ft})) + (2 \times (-0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.0002891 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0084395 \text{ kipft/ft})) + ((-0.0019108 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 0.0024534 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.1517 \text{ ft})}{2}$$

$$p_a = 0.63034 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0002891 \text{ kip/ft}^2)}{(0.63034 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00045864$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$$

$$p_s = 1.8 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

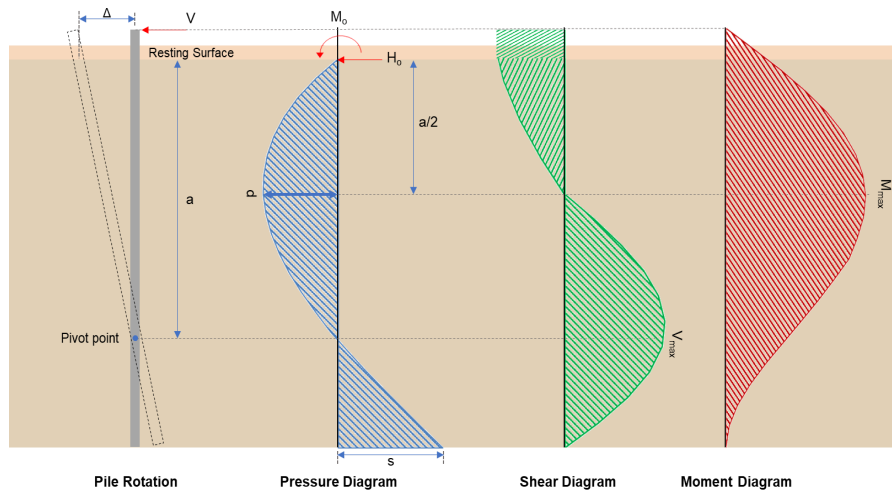
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0024534 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$Ratio = 0.001363$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.576 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.41019 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.378 \text{ kipft}) + ((-2.576 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6334 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6334 \text{ kipft/ft})}{(-0.41019 \text{ kip/ft})}$$

$$E = 13.734 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6334 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.41019 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (5.6334 \text{ kipft/ft})) + (4 \times (-0.41019 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0672 \text{ ft}$$

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

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

$$V_{max} = ((-0.41019 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0672 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0672 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.9518 \text{ kip}$$

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

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

$$M_{max} = ((-0.41019 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(13.734 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (13.734 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0672 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.717 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.019 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0030255 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.082 \text{ kipft}) + ((-0.019 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.013057 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.013057 \text{ kipft/ft})}{(-0.0030255 \text{ kip/ft})}$$

$$E = 4.3158 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.013057 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.0030255 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.013057 \text{ kipft/ft})) + (4 \times (-0.0030255 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.1538 \text{ ft}$$

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

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

$$V_{max} = ((-0.0030255 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1538 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1538 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.028534 \text{ kip}$$

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

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

$$M_{max} = ((-0.0030255 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.3158 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.3158 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.1538 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.059298 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.188 \text{ kip})}{(0.65)(0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.357 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.357 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(7.188 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0026869$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.188 \text{ kip} \rightarrow 7188 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7188 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.44 \text{ kip}$$

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

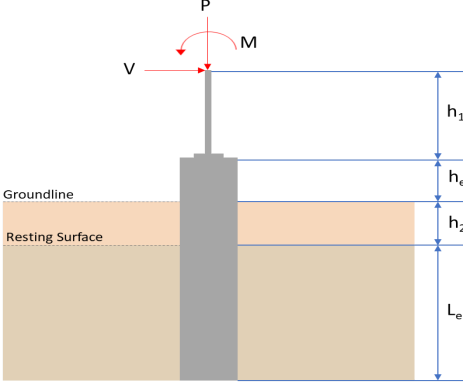
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.44 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.44 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.44 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.72 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 9.9518 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(9.9518 \text{ kip})}{(110.72 \text{ kip})}$ $\text{Ratio} = 0.089883$ <p>Considering z-direction:</p> <p>$V_{max} = 0.028534 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.028534 \text{ kip})}{(110.72 \text{ kip})}$ $\text{Ratio} = 0.00025771$	<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: $M_{max} = 21.717 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(21.717 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.087006$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.059298 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.059298 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00023757$	<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 $L = 4.5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="512 1102 1098 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>Sedimentary & foliated rock</td> <td>4000.000</td> <td>400.000</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>4.920</td> <td>7.287</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.604</td> <td>-2.674</td> </tr> <tr> <td>V_z (kip)</td> <td>0.107</td> <td>0.156</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.436</td> <td>0.636</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.958</td> <td>37.297</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	Sedimentary & foliated rock	4000.000	400.000	Load Component	ASD	LRFD	P (kip)	4.920	7.287	V_x (kip)	-1.604	-2.674	V_z (kip)	0.107	0.156	M_x (kipft)	0.436	0.636	M_z (kipft)	21.958	37.297	
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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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.604 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25541 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.958 \text{ kipft}) + ((-1.604 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.4965 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x}$ = 4.311 ft - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.107 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.017038 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.436 \text{ kipft}) + ((0.107 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.069427 \text{ kipft/ft}$$

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:

$L_{e,z}$ = 1.377 ft - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.311 \text{ ft}), (1.377 \text{ ft})]$$

$$L_{e,req} = 4.311 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.311 \text{ ft})}{(4.5 \text{ ft})}$$

$$\text{Ratio} = 0.958$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.92 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.3075 \text{ kip/ft}^2$$

$$q = 0.3075 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.3075 \text{ kip/ft}^2)}{(4000 \text{ psf})}$$

$$\text{Ratio} = 0.076875$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25541 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.4965 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4965 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (3.4965 \text{ kipft/ft})) + (4 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0674 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (3.4965 \text{ kipft/ft})) + (3 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (3.4965 \text{ kipft/ft})) + (2 \times (-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$$

$$p = 0.50213 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (3.4965 \text{ kipft/ft})) + ((-0.25541 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$$

$$s = 1.7314 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (400 \text{ psf/ft}) \times \frac{(3.0674 \text{ ft})}{2}$$

$$p_a = 0.61348 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.50213 \text{ kip/ft}^2)}{(0.61348 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8185$$

p_a - Allowable lateral soil pressure at depth L_e ,

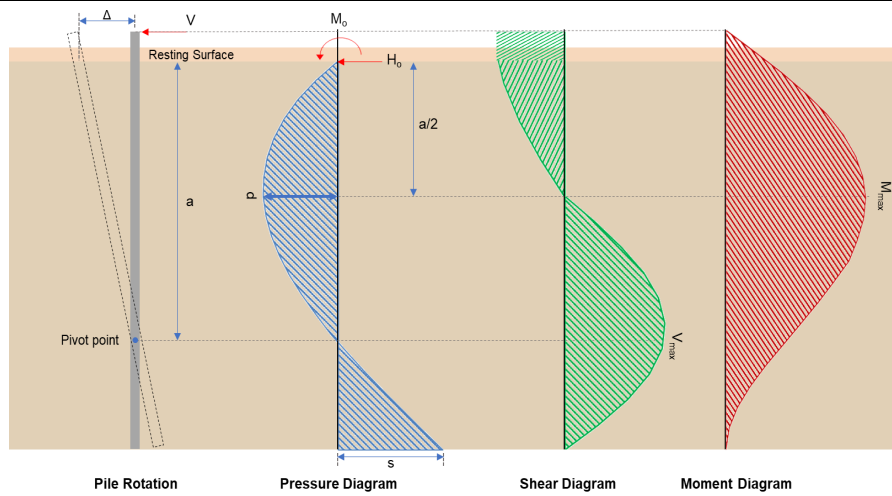
Status: **PASS**
Ratio: **0.820**

	$p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(1.7314 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.96191$	<p>Status: PASS Ratio: 0.960</p>
	<p>Considering z-direction:</p> <p>$H_o = 0.017038 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.069427 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.069427 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.017038 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.069427 \text{ kipft/ft})) + (4 \times (0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))}$ $a = 3.159 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.069427 \text{ kipft/ft})) + (3 \times (0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]^2}{(4.5 \text{ ft})^2 [(3 \times (0.069427 \text{ kipft/ft})) + (2 \times (0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]}$ $p = 0.026402 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.069427 \text{ kipft/ft})) + ((0.017038 \text{ kip/ft}) \times (4.5 \text{ ft}))]}{(4.5 \text{ ft})^2}$ $s = 0.063859 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (400 \text{ psf/ft}) \times \frac{(3.159 \text{ ft})}{2}$ $p_a = 0.6318 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.026402 \text{ kip/ft}^2)}{(0.6318 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.041788$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (400 \text{ psf/ft}) \times (4.5 \text{ ft})$ $p_s = 1.8 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: 0.040</p>

$$\text{Ratio} = \frac{(0.063859 \text{ kip/ft}^2)}{(1.8 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.035477$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.674 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.4258 \text{ kip/ft}$$

M_o - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(37.297 \text{ kipft}) + ((-2.674 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.939 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.939 \text{ kipft/ft})}{(-0.4258 \text{ kip/ft})}$$

$$E = 13.948 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.939 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (-0.4258 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (5.939 \text{ kipft/ft})) + (4 \times (-0.4258 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.0664 \text{ ft}$$

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

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

$$V_{max} = ((-0.4258 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0664 \text{ ft})}{(4.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0664 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.474 \text{ kip}$$

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

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

$$M_{max} = ((-0.4258 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(13.948 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (13.948 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.0664 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 22.867 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.156 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.024841 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.636 \text{ kipft}) + ((0.156 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.10127 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.10127 \text{ kipft/ft})}{(0.024841 \text{ kip/ft})}$$

$$E = 4.0769 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.10127 \text{ kipft/ft}) \times (4.5 \text{ ft})) + (3 \times (0.024841 \text{ kip/ft}) \times (4.5 \text{ ft})^2)}{(6 \times (0.10127 \text{ kipft/ft})) + (4 \times (0.024841 \text{ kip/ft}) \times (4.5 \text{ ft}))}$$

$$a = 3.159 \text{ ft}$$

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

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

$$V_{max} = ((0.024841 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.159 \text{ ft})}{(4.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.159 \text{ ft})}{(4.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.22498 \text{ kip}$$

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

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

$$M_{max} = ((0.024841 \text{ kip/ft}) \times (48 \text{ in}) \times (4.5 \text{ ft})) \times \left[\left(\frac{(4.0769 \text{ ft})}{(4.5 \text{ ft})} + \frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 3 \right) \times \left(\frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (4.0769 \text{ ft})}{(4.5 \text{ ft})} + 2 \right) \times \left(\frac{(3.159 \text{ ft})}{2 \times (4.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.46598 \text{ kipft}$$

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.287 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.354 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.354 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

$$\text{Ratio} = \frac{(7.287 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0027239$$

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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.287 \text{ kip} \rightarrow 7287 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7287 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.46 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.46 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.46 \text{ kip}$$

<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> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.46 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.73 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 10.474 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(10.474 \text{ kip})}{(110.73 \text{ kip})}$ $\text{Ratio} = 0.094595$ <p>Considering z-direction:</p> <p>$V_{max} = 0.22498 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.22498 \text{ kip})}{(110.73 \text{ kip})}$ $\text{Ratio} = 0.0020318$	<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: $M_{max} = 22.867 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(22.867 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.091613$	<p>Status: PASS Ratio: 0.090</p>
	<p>Considering z-direction: $M_{max} = 0.46598 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.46598 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.0018669$	<p>Status: PASS Ratio: 0.000</p>