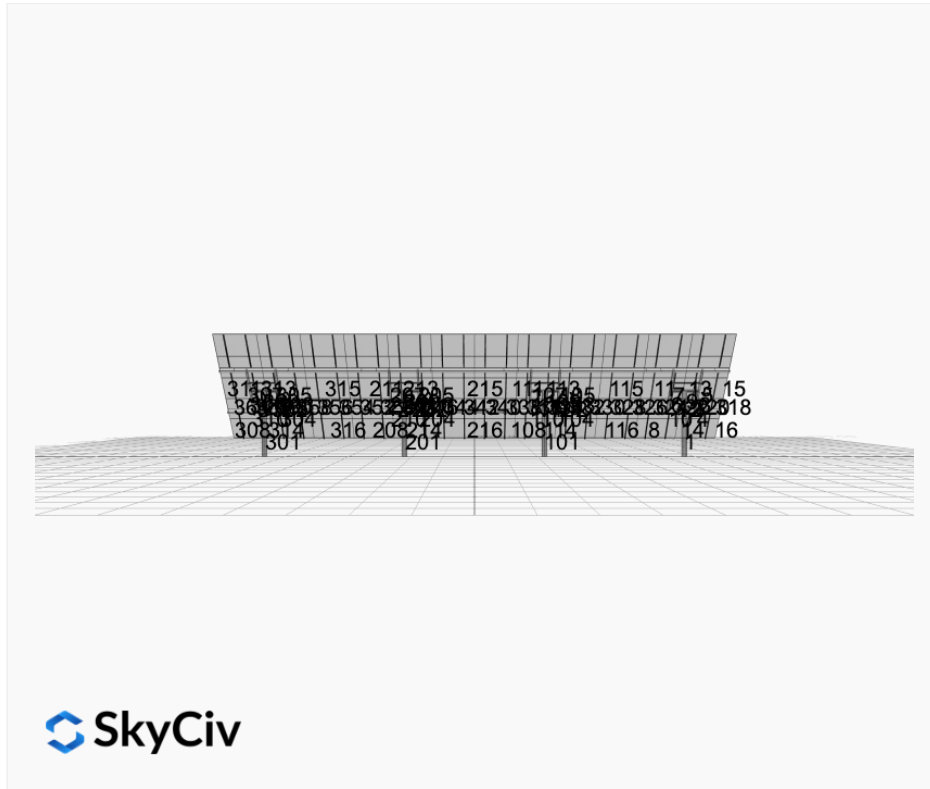


Project Name: MTSOLAR_C3H4D1F10J3K 5x12 - V1Jb
Date: Mon Aug 04 2025
Location: Franconia, NH, USA
Number of Modules: 60
Unique ID: 4P-19.75-8TOP-HD-24-L-5Hx12W-KC31
Number of Poles: 4
Date Sold:
Dealer: _____



Array Dimensions N/S	18.96 ft
Array Dimensions E/W	71.00 ft
Winter Tilt Angle	50
Front Edge Clearance	2 ft

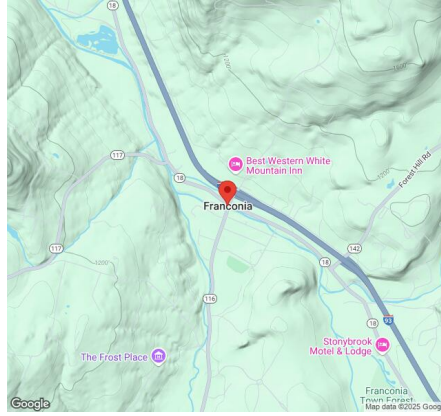
MT Solar Bill of Materials (4P-19.75-8TOP-HD-24-L-5Hx12W-KC31)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-HD	H-Frame Assembly-HD	4
MTS-HD-Wing-24	24IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	6
MTS-HD-Splice-57	57IN HD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

Rail Bill of Materials

Part	Qty
Rails (228in)	24
Rail Attachment	96
Module Mid Clamp	96
Module End Clamp	48
Ground Lug	12

Site Details:



Site Address: Franconia, NH, USA

Array Specification

Duty Classification:	HD
Module Width:	45.00 in
Module Length:	70.00in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Winter Tilt Angle:	50
Front Edge Clearance:	2
Total Array Height at Tilt:	16.52 ft
Total Frame Length:	70.75 ft
Module Info/Notes:	Trina 420
Array Dimensions N/S:	18.96 ft
Array Dimensions E/W:	71.00 ft
Rail Length:	227.50 in
Rail Spacing:	2.96 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	9.26 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.75 ft Pile 3: 6.75 ft Pile 4: 6.25 ft
Foundation Volume:	15.407 y ³

Site Info

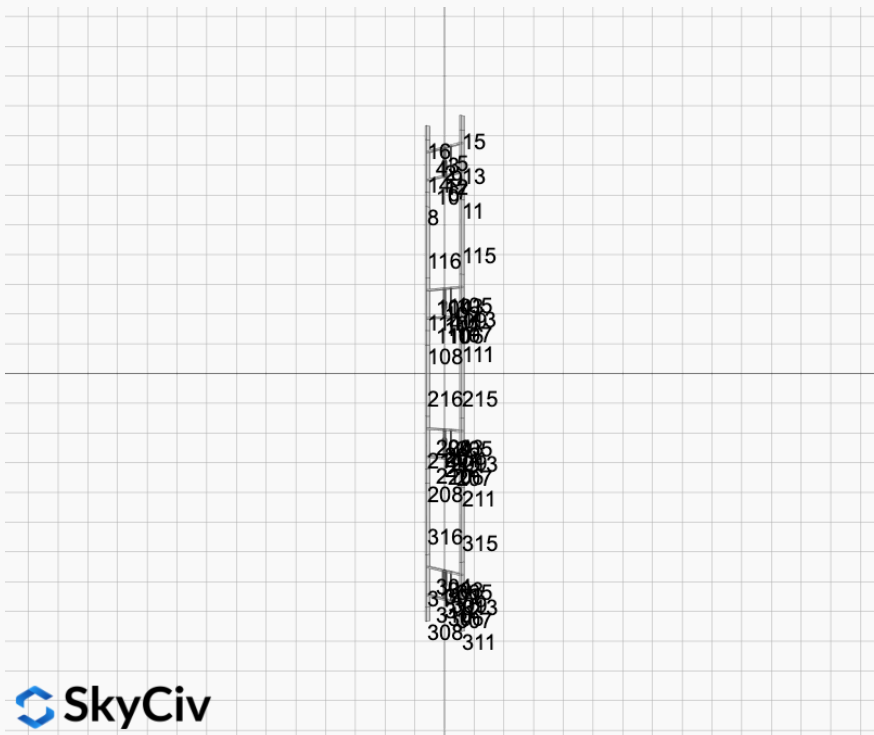
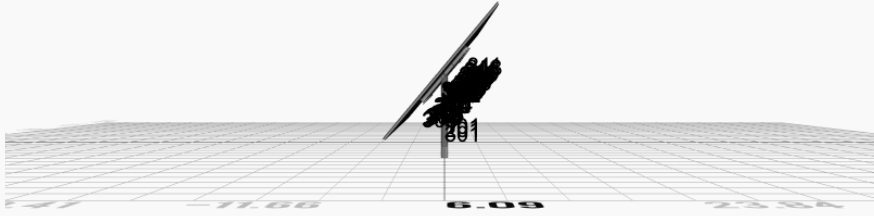
Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Franconia, NH, USA
Wind Speed:	106 mph

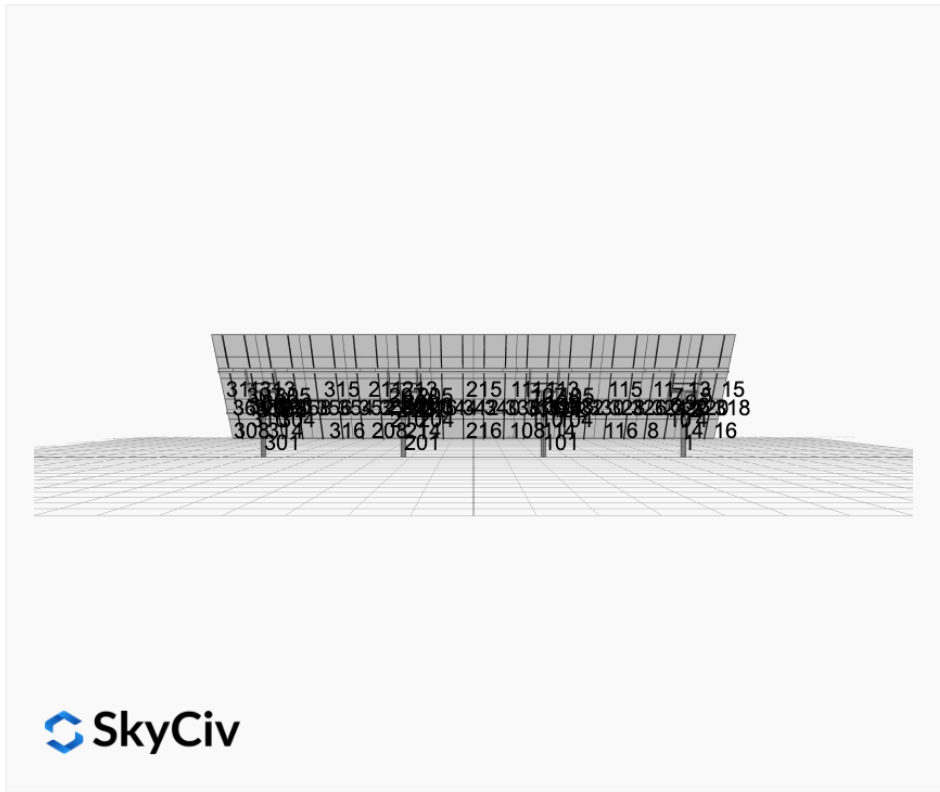
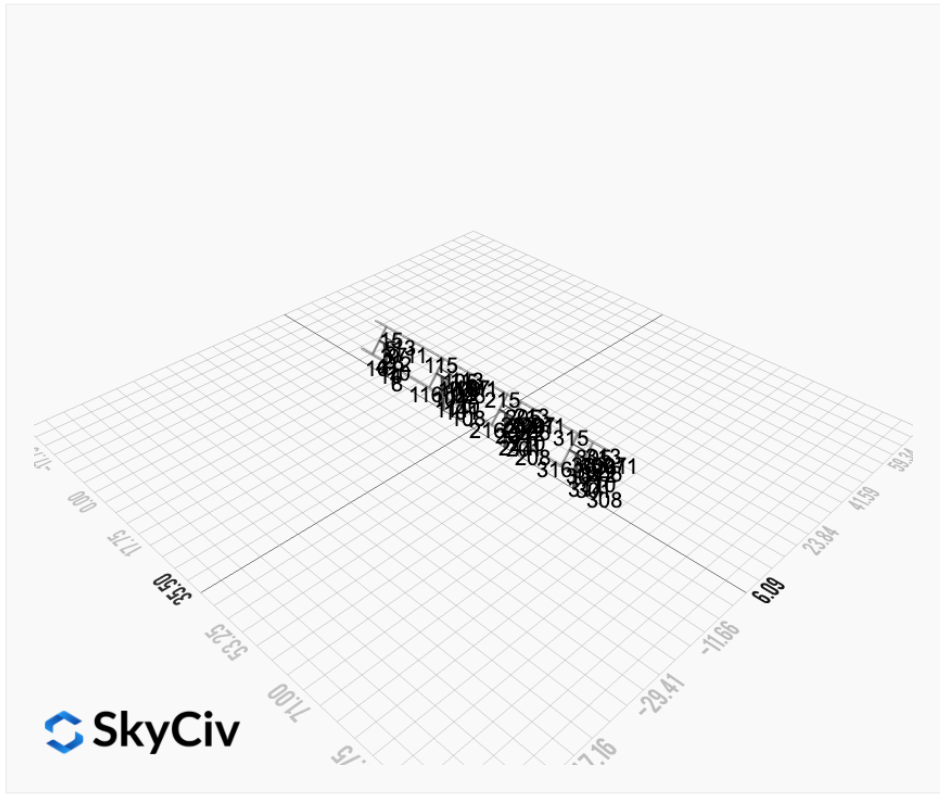
Snow Load:

90 psf

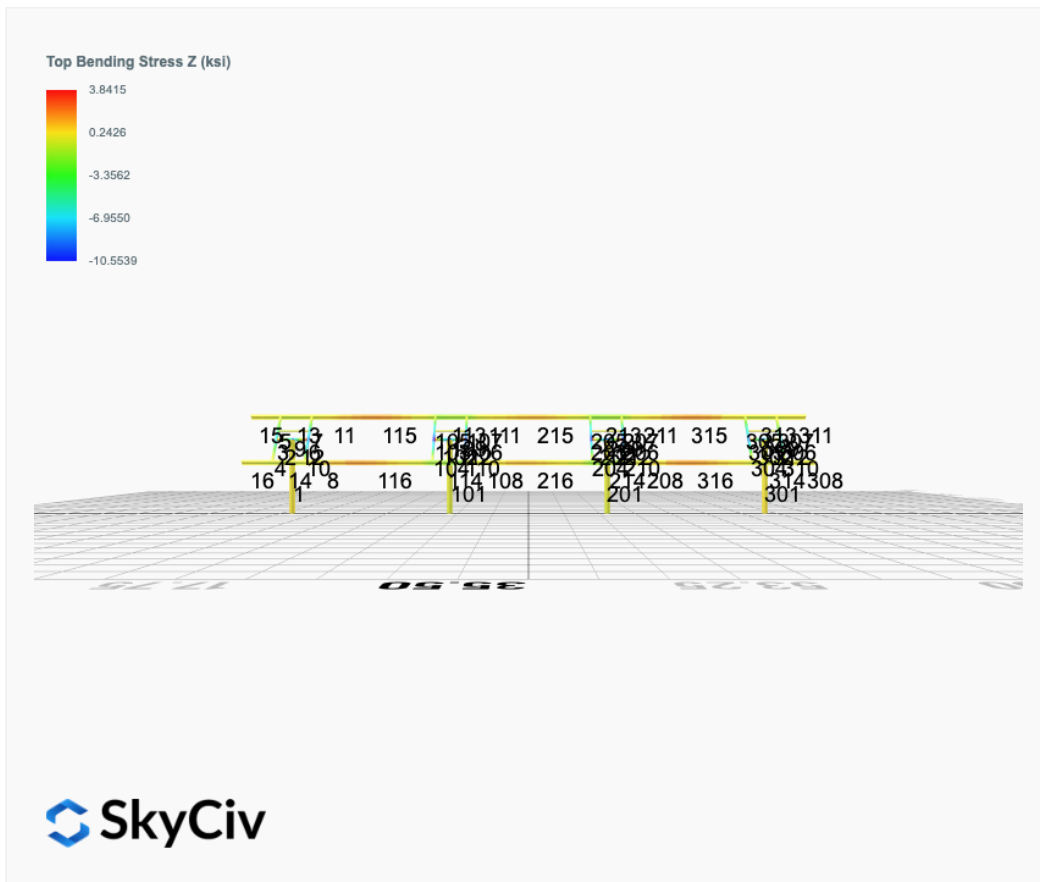
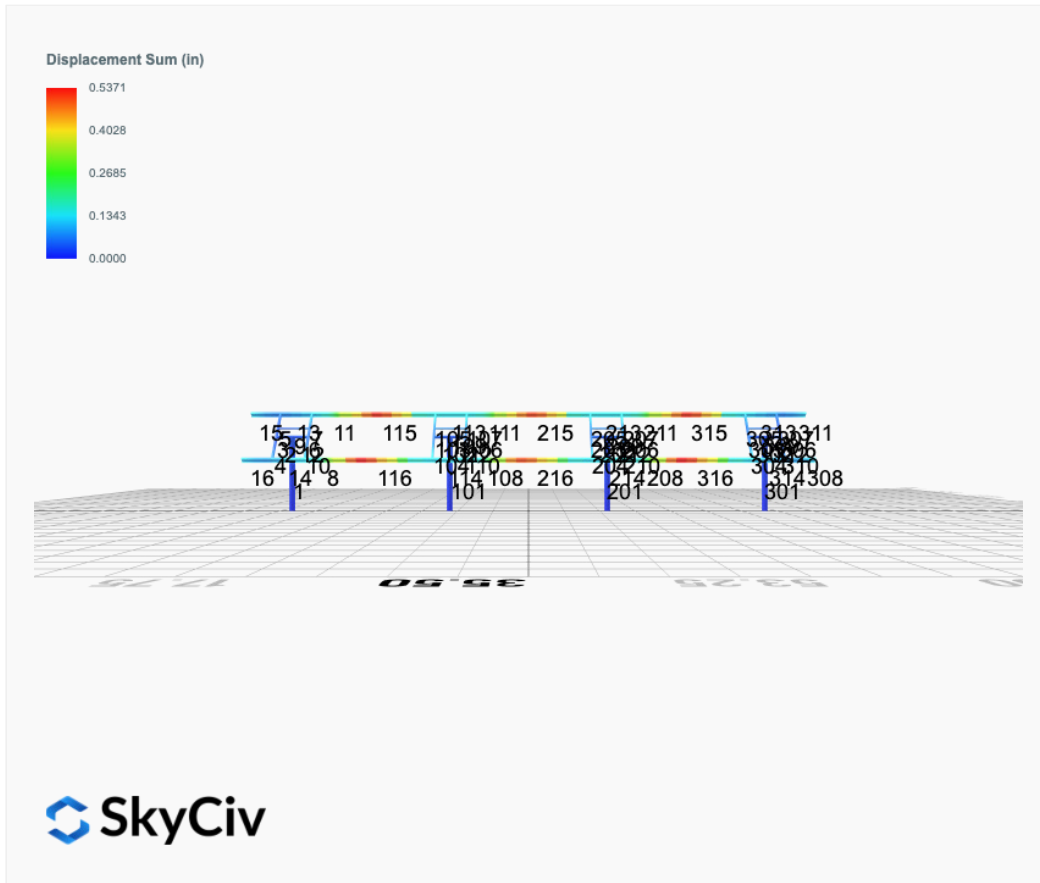
Design Disclaimer

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.

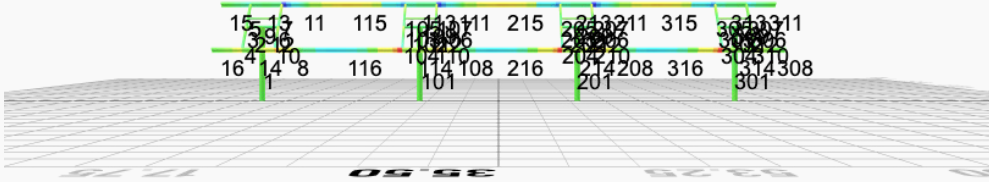
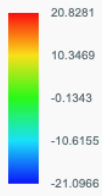




FEM Results (Envelope Worst Case for each member)



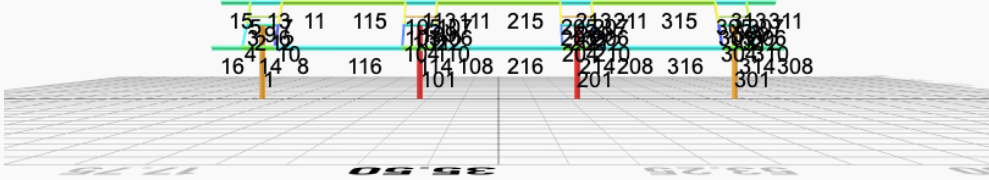
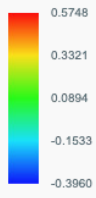
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 2. D + L	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 3. D + (S or Lr or R)	0.1016	5.8159	0.3056	0.7540	-0.3007	-0.8627
ULS: 3. D + (S or Lr or R)	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0840	4.8897	0.2525	0.6229	-0.2484	-0.7106
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 5b. D + 0.7E	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0840	4.8897	0.2525	0.6229	-0.2484	-0.7106
ULS: 8. 0.6D + 0.7E	0.0187	1.2667	0.0559	0.1376	-0.0550	-0.1526
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8210	4.4769	0.3226	0.7321	-1.1942	26.5654
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8815	-0.2537	-0.1331	-0.2662	0.9978	-26.6364
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0551	6.6640	0.4245	0.9999	-1.0754	19.4041
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0840	4.8897	0.2525	0.6229	-0.2484	-0.7106
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2218	3.1161	0.0828	0.2512	0.5686	-20.4972
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0840	4.8897	0.2525	0.6229	-0.2484	-0.7106
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1080	3.8855	0.2653	0.6064	-0.9186	19.8604
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1689	0.3375	-0.0765	-0.1423	0.7254	-20.0409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0311	2.1111	0.0932	0.2294	-0.0916	-0.2543
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8335	3.6325	0.2854	0.6403	-1.1576	26.6671
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0187	1.2667	0.0559	0.1376	-0.0550	-0.1526
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8691	-1.0981	-0.1704	-0.3580	1.0344	-26.5347
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0187	1.2667	0.0559	0.1376	-0.0550	-0.1526

Worst Case Reactions LRFD

These calculations are taken directly from the FEA via SkyCiv and are used in the Concrete Checks of the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	10.4328
Shear X	-4.8245
Shear Z	0.6477
Moment X	1.5514
Moment Y (Twist)	2.0617
Moment Z	45.0163

Worst Case Reactions ASD

These results are taken from the worst case values in the above table and are used in the Soil Checks in the Foundation Module.

Note: Worst case values are assumed as downforce wind load cases.

Result	Value (kip, kip-ft)
Axial	6.6640
Shear X	-2.8815
Shear Z	0.4245
Moment X	0.9999
Moment Y (Twist)	1.1942
Moment Z	26.6671

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	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 2. D + L	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 3. D + (S or Lr or R)	-0.1016	7.4345	-0.0173	-0.0444	0.0797	0.9559
ULS: 3. D + (S or Lr or R)	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0840	6.2275	-0.0143	-0.0367	0.0659	0.7913

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 5b. D + 0.7E	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0840	6.2275	-0.0143	-0.0367	0.0659	0.7913
ULS: 8. 0.6D + 0.7E	-0.0187	1.5639	-0.0032	-0.0082	0.0147	0.1787
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.8058	5.8013	-0.0010	-0.0091	-0.0336	35.4266
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7453	-0.5892	-0.0082	-0.0151	0.0749	-34.1326
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.9150	8.6236	-0.0111	-0.0333	0.0223	27.1380
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0840	6.2275	-0.0143	-0.0367	0.0659	0.7913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7483	3.8307	-0.0165	-0.0378	0.1037	-25.0315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0840	6.2275	-0.0143	-0.0367	0.0659	0.7913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8621	5.0026	-0.0021	-0.0102	-0.0191	26.6444
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8012	0.2097	-0.0075	-0.0148	0.0623	-25.5250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0311	2.6066	-0.0053	-0.0136	0.0244	0.2978
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7933	4.7587	0.0011	-0.0036	-0.0434	35.3075
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0187	1.5639	-0.0032	-0.0082	0.0147	0.1787
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7577	-1.6318	-0.0060	-0.0097	0.0652	-34.2517
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0187	1.5639	-0.0032	-0.0082	0.0147	0.1787

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	13.5145
Shear X	-6.3623
Shear Z	-0.0293
Moment X	-0.0695
Moment Y (Twist)	0.1673
Moment Z	59.9740

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.6236
Shear X	-3.8058
Shear Z	-0.0173
Moment X	-0.0444
Moment Y (Twist)	0.1037
Moment Z	35.4266

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	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 2. D + L	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 3. D + (S or Lr or R)	-0.1016	7.4345	0.0174	0.0442	-0.0794	0.9558
ULS: 3. D + (S or Lr or R)	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0840	6.2275	0.0144	0.0366	-0.0656	0.7913
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 5b. D + 0.7E	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0840	6.2275	0.0144	0.0366	-0.0656	0.7913
ULS: 8. 0.6D + 0.7E	-0.0187	1.5639	0.0032	0.0082	-0.0146	0.1787
ULS: 5a. D + 0.6W_Wind downforce Case A only	-3.8058	5.8013	0.0010	0.0091	0.0337	35.4266
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 5a. D + 0.6W_Wind uplift Case A only	3.7453	-0.5892	0.0082	0.0151	-0.0749	-34.1326
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.9150	8.6236	0.0112	0.0332	-0.0221	27.1380
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0840	6.2275	0.0144	0.0366	-0.0656	0.7913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.7483	3.8307	0.0165	0.0377	-0.1035	-25.0315
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0840	6.2275	0.0144	0.0366	-0.0656	0.7913
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.8621	5.0026	0.0021	0.0102	0.0192	26.6444
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.8012	0.2097	0.0075	0.0147	-0.0623	-25.5250
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0311	2.6066	0.0053	0.0136	-0.0244	0.2978
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-3.7933	4.7587	-0.0011	0.0036	0.0434	35.3075
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0187	1.5639	0.0032	0.0082	-0.0146	0.1787
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	3.7577	-1.6318	0.0060	0.0097	-0.0652	-34.2517
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0187	1.5639	0.0032	0.0082	-0.0146	0.1787

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	13.5145
Shear X	-6.3623
Shear Z	0.0294
Moment X	0.0694
Moment Y (Twist)	0.1664
Moment Z	59.9741

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.6236
Shear X	-3.8058
Shear Z	0.0174
Moment X	0.0442
Moment Y (Twist)	0.1035
Moment Z	35.4266

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	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 2. D + L	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 3. D + (S or Lr or R)	0.1016	5.8158	-0.3056	-0.7545	0.3010	-0.8626
ULS: 3. D + (S or Lr or R)	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0840	4.8896	-0.2525	-0.6233	0.2486	-0.7105
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 5b. D + 0.7E	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0840	4.8896	-0.2525	-0.6233	0.2486	-0.7105
ULS: 8. 0.6D + 0.7E	0.0187	1.2667	-0.0559	-0.1377	0.0550	-0.1526
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.8210	4.4769	-0.3226	-0.7321	1.1942	26.5654
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.8815	-0.2537	0.1331	0.2662	-0.9977	-26.6364
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.0551	6.6640	-0.4246	-1.0003	1.0756	19.4043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0840	4.8896	-0.2525	-0.6233	0.2486	-0.7105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.2218	3.1160	-0.0828	-0.2516	-0.5684	-20.4970
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0840	4.8896	-0.2525	-0.6233	0.2486	-0.7105
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-2.1080	3.8855	-0.2653	-0.6065	0.9186	19.8605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	2.1689	0.3375	0.0765	0.1423	-0.7254	-20.0409
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0311	2.1111	-0.0932	-0.2295	0.0916	-0.2543

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.8335	3.6325	-0.2854	-0.6403	1.1576	26.6671
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0187	1.2667	-0.0559	-0.1377	0.0550	-0.1526
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.8691	-1.0981	0.1704	0.3579	-1.0344	-26.5346
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0187	1.2667	-0.0559	-0.1377	0.0550	-0.1526

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	10.4327
Shear X	-4.8245
Shear Z	-0.6477
Moment X	-1.5538
Moment Y (Twist)	2.0624
Moment Z	45.0168

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	6.6640
Shear X	-2.8815
Shear Z	-0.4246
Moment X	-1.0003
Moment Y (Twist)	1.1942
Moment Z	26.6671

Project Details

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

 User Name: sales@mtsolar.us
 Project Name: MTSOLAR_C3H4D1F10J3K 5x12 - V1Jb
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t_w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
9	8in Pipe Sch 40	8.63	0.32				

ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
16	HSS5x3x3/16	5.00	3.00	0.17	0.17	0.17	

ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
19	W8x10	7.89	0.17	3.94	3.94	0.20	0.20	0.30

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I_{y0} (in ⁴)	I_{z0} (in ⁴)	I_w (in ⁶)	S_{y0} (in ³)	S_{z0} (in ³)

2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	0.73	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties									
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	LS T	LS C	L D	
1	9	19.45	19.45	9.26	-	300	200	1	
2	5	1.30	1.30	2.00	-	300	200	1	
3	16	0.92	0.92	1.42	1.18,1.18,1.18,1.17,1.18,1.18,1.17,1.18,1.16,1.18,1.17,1.18,1.17,1.18,1.17,1.17,1.18,1.17,1.17,1.18,1.1	300	200	1	6,1.18,1.17,1.18,1.17,1.18
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.66,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.69,1.6	300	200	1	5,1.69,1.67,1.69,1.66,1.69
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.66,1.68
6	16	0.92	0.92	1.42	1.19,1.19,1.19,1.18,1.19,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.18,1.19,1.19,1.1	300	200	1	8,1.19,1.18,1.19,1.18,1.19
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.67,1.68
8	19	1.33	1.33	2.05	1.32,1.32,1.32,1.32,1.32,1.32,1.31,1.32,1.29,1.32,1.30,1.32,1.30,1.32,1.31,1.32,1.35,1.32,1.31,1.32,1.2	300	200	1	9,1.32,1.30,1.32,1.30,1.32
9	2	2.60	2.60	4.00	-	300	200	1	
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.66,1.68
11	19	1.33	1.33	2.05	1.34,1.34,1.34,1.34,1.34,1.34,1.38,1.34,1.42,1.34,1.38,1.34,1.40,1.34,1.36,1.34,1.28,1.34,1.38,1.34,1.4	300	200	1	2,1.34,1.38,1.34,1.40,1.34
12	5	1.30	1.30	2.00	-	300	200	1	
13	19	4.88	4.00	7.50	1.17,1.17,1.17,1.17,1.17,1.17,1.15,1.17,1.13,1.17,1.15,1.17,1.14,1.17,1.16,1.17,1.22,1.17,1.15,1.17,1.1	300	200	1	3,1.17,1.15,1.17,1.14,1.17
14	19	4.88	4.00	7.50	1.17,1.17,1.17,1.17,1.17,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.18,1.17,1.17,1.17,1.16,1.17,1.17,1.17,1.1	300	200	1	8,1.17,1.18,1.17,1.18,1.17
15	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1	3,2.33,2.33,2.33,2.33,2.33
16	19	4.20	4.20	2.00	2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.3	300	200	1	3,2.33,2.33,2.33,2.33,2.33
101	9	19.45	19.45	9.26	-	300	200	1	
102	5	1.30	1.30	2.00	-	300	200	1	
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.1	300	200	1	7,1.19,1.18,1.19,1.18,1.19
104	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.66,1.68
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.66,1.68
106	16	0.92	0.92	1.42	1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.1	300	200	1	7,1.18,1.18,1.18,1.18,1.18
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.67,1.68
108	19	1.33	1.33	2.05	2.14,2.16,2.14,2.16,2.15,2.14,2.11,2.16,2.10,2.16,2.11,2.14,2.11,2.14,2.12,2.16,2.39,2.16,2.11,2.14,2.1	300	200	1	0,2.14,2.11,2.14,2.11,2.14
109	2	2.60	2.60	4.00	-	300	200	1	
110	16	2.44	2.44	3.75	1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.6	300	200	1	6,1.68,1.67,1.68,1.66,1.68
111	19	1.33	1.33	2.05	2.10,2.10,2.10,2.10,2.10,2.10,2.06,2.10,1.72,2.10,2.03,2.10,1.86,2.10,2.07,2.10,2.21,2.10,2.06,2.10,1.7	300	200	1	1,2.10,2.02,2.10,1.89,2.10
112	5	1.30	1.30	2.00	-	300	200	1	

212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.69	6.12	40.24	43.62
214	133.20	85.85	23.76	6.12	40.24	43.62
215	133.20	69.16	17.42	6.12	40.24	43.62
216	133.20	69.16	18.01	6.12	40.24	43.62
301	377.97	238.25	83.29	83.29	113.39	113.39
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	102.39	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	102.39	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	85.85	25.91	6.12	40.24	43.62
314	133.20	85.85	26.65	6.12	40.24	43.62
315	133.20	69.16	16.82	6.12	40.24	43.62
316	133.20	69.16	16.78	6.12	40.24	43.62

Design Ratio

Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.044	0.541	0.053	0.043	0.006	0.569	#13	0.397	Not Required	Pass
2	0.004	0.243	0.163	0.063	0.035	0.358	#13	0.053	Not Required	Pass
3	0.010	0.421	0.040	0.041	0.003	0.433	#13	0.045	Not Required	Pass
4	0.008	0.422	0.117	0.042	0.029	0.511	#21	0.080	Not Required	Pass
5	0.009	0.262	0.090	0.042	0.023	0.270	#21	0.074	Not Required	Pass
6	0.017	0.623	0.163	0.064	0.043	0.752	#21	0.045	Not Required	Pass
7	0.018	0.387	0.277	0.062	0.068	0.427	#21	0.074	Not Required	Pass
8	0.006	0.121	0.245	0.038	0.028	0.266	#21	0.095	Not Required	Pass
9	0.010	0.051	0.104	0.005	0.006	0.120	#13	0.204	Not Required	Pass
10	0.019	0.594	0.258	0.060	0.055	0.709	#21	0.080	Not Required	Pass
11	0.009	0.112	0.256	0.041	0.028	0.273	#21	0.095	Not Required	Pass
12	0.002	0.445	0.257	0.107	0.050	0.642	#13	0.053	Not Required	Pass
13	0.012	0.108	0.654	0.053	0.036	0.721	#21	0.286	Not Required	Pass
14	0.008	0.098	0.638	0.051	0.036	0.679	#21	0.190	Not Required	Pass
15	0.000	0.017	0.064	0.014	0.009	0.081	#21	Not Required	Not Required	Pass
16	0.000	0.017	0.064	0.014	0.009	0.081	#21	Not Required	Not Required	Pass
101	0.057	0.720	0.002	0.056	0.000	0.743	#13	0.397	Not Required	Pass
102	0.005	0.456	0.289	0.112	0.053	0.676	#13	0.035	Not Required	Pass
103	0.017	0.682	0.105	0.068	0.015	0.766	#21	0.045	Not Required	Pass
104	0.017	0.698	0.273	0.070	0.058	0.849	#21	0.080	Not Required	Pass
105	0.017	0.423	0.288	0.068	0.072	0.480	#21	0.074	Not Required	Pass
106	0.017	0.691	0.104	0.069	0.015	0.761	#21	0.045	Not Required	Pass
107	0.017	0.430	0.281	0.069	0.071	0.480	#21	0.074	Not Required	Pass
108	0.006	0.046	0.241	0.043	0.028	0.285	#21	0.095	Not Required	Pass
109	0.023	0.043	0.064	0.001	0.000	0.107	#13	0.204	Not Required	Pass
110	0.017	0.682	0.269	0.060	0.057	0.825	#21	0.080	Not Required	Pass

110	0.017	0.069	0.206	0.009	0.037	0.633	#21	0.000	Not Required	Pass
111	0.008	0.063	0.251	0.043	0.028	0.287	#21	0.095	Not Required	Pass
112	0.005	0.450	0.291	0.110	0.054	0.677	#13	0.035	Not Required	Pass
113	0.012	0.195	0.664	0.058	0.036	0.833	#21	0.286	Not Required	Pass
114	0.011	0.229	0.653	0.060	0.036	0.842	#21	0.286	Not Required	Pass
115	0.015	0.324	0.363	0.046	0.028	0.669	#21	0.473	Not Required	Pass
116	0.006	0.311	0.357	0.048	0.028	0.648	#21	0.473	Not Required	Pass
201	0.057	0.720	0.002	0.056	0.000	0.743	#13	0.397	Not Required	Pass
202	0.005	0.450	0.291	0.110	0.054	0.677	#13	0.035	Not Required	Pass
203	0.017	0.691	0.104	0.069	0.015	0.761	#21	0.045	Not Required	Pass
204	0.017	0.689	0.268	0.069	0.057	0.835	#21	0.080	Not Required	Pass
205	0.017	0.430	0.281	0.069	0.071	0.480	#21	0.074	Not Required	Pass
206	0.017	0.682	0.105	0.068	0.015	0.766	#21	0.045	Not Required	Pass
207	0.017	0.423	0.288	0.068	0.072	0.480	#21	0.074	Not Required	Pass
208	0.006	0.073	0.256	0.048	0.028	0.286	#21	0.095	Not Required	Pass
209	0.023	0.043	0.064	0.001	0.000	0.107	#13	0.204	Not Required	Pass
210	0.017	0.698	0.273	0.070	0.058	0.849	#21	0.080	Not Required	Pass
211	0.009	0.088	0.264	0.046	0.028	0.284	#21	0.095	Not Required	Pass
212	0.005	0.456	0.289	0.112	0.053	0.676	#13	0.035	Not Required	Pass
213	0.012	0.195	0.664	0.058	0.036	0.834	#21	0.286	Not Required	Pass
214	0.011	0.229	0.653	0.060	0.036	0.842	#21	0.286	Not Required	Pass
215	0.015	0.250	0.363	0.043	0.028	0.593	#21	0.473	Not Required	Pass
216	0.006	0.214	0.356	0.043	0.028	0.557	#21	0.473	Not Required	Pass
301	0.044	0.541	0.053	0.043	0.006	0.569	#13	0.397	Not Required	Pass
302	0.002	0.445	0.257	0.107	0.050	0.642	#13	0.053	Not Required	Pass
303	0.017	0.623	0.163	0.064	0.043	0.752	#21	0.045	Not Required	Pass
304	0.019	0.594	0.258	0.060	0.055	0.709	#21	0.080	Not Required	Pass
305	0.018	0.387	0.277	0.062	0.068	0.427	#21	0.074	Not Required	Pass
306	0.010	0.421	0.040	0.041	0.003	0.433	#13	0.045	Not Required	Pass
307	0.009	0.262	0.090	0.042	0.023	0.270	#21	0.074	Not Required	Pass
308	0.000	0.017	0.064	0.014	0.009	0.081	#21	Not Required	Not Required	Pass
309	0.010	0.051	0.105	0.005	0.006	0.120	#13	0.204	Not Required	Pass
310	0.008	0.422	0.117	0.042	0.029	0.511	#21	0.080	Not Required	Pass
311	0.000	0.017	0.064	0.014	0.009	0.081	#21	Not Required	Not Required	Pass
312	0.004	0.243	0.163	0.063	0.035	0.358	#13	0.053	Not Required	Pass
313	0.012	0.108	0.653	0.053	0.036	0.720	#21	0.190	Not Required	Pass
314	0.008	0.097	0.639	0.051	0.036	0.679	#21	0.286	Not Required	Pass
315	0.015	0.333	0.363	0.041	0.028	0.678	#21	0.473	Not Required	Pass
316	0.006	0.332	0.356	0.038	0.028	0.661	#21	0.473	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength
A	Cross-sectional area
J	Torsional constant
I_{yp}	Moment of inertia about the Y axes
I_{zp}	Moment of inertia about the Z axes
I_w	Warping constant
S_{yp}	Plastic section modulus about the Y axis

S_{zp}	Plastic section modulus about the Z axis
KL	Effective length
C_b	Buckling modification factor (from all load combinations)
L_b	Length between braced points
LST	Limited slenderness for tension
LSC	Limited slenderness for compression
LD	Limited deflection
P_n	Nominal axial strength (tension/compression)
M_n	Nominal flexural strength (about Z/Y axis)
V_n	Nominal shear strength (along Z/Y axis)
P	Design ratio in case of axial force
M_z	Design ratio in case of bending about Z axis
M_y	Design ratio in case of bending about Y axis
V_y	Design ratio in case of shear along Y axis
V_z	Design ratio in case of shear along Z axis
(P, M_z, M_y)	Design ratio in case of axial force and bending action
KL/r	Design ratio in case of section slenderness
δ	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided

REFERENCES	CALCULATIONS	RESULTS
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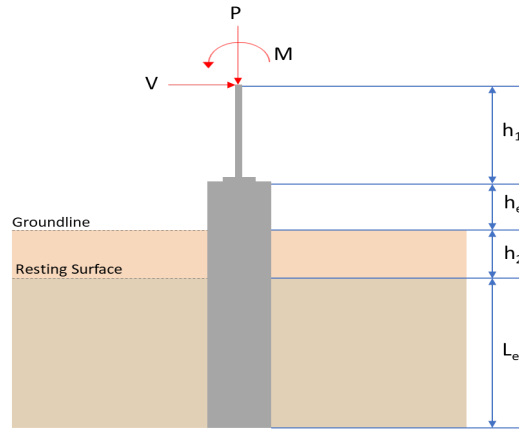
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.664	10.433
V_x (kip)	-2.882	-4.825
V_z (kip)	0.425	0.648
M_x (kipft)	1.000	1.551
M_z (kipft)	26.667	45.016

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.882 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45892 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.667 \text{ kipft}) + ((-2.882 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.2463 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6805 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.425 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.067675 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1 \text{ kipft}) + ((0.425 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15924 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.906 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6805 \text{ ft}), (2.906 \text{ ft})]$$

$$L_{e,req} = 5.68 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.68 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9088$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.664 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.4165 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4165 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20825$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.45892 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.2463 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2463 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.2463 \text{ kipft/ft})) + (4 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3284 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.2463 \text{ kipft/ft})) + (3 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.2463 \text{ kipft/ft})) + (2 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.19258 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.2463 \text{ kipft/ft})) + ((-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3284 \text{ ft})}{2}$$

$$p_a = 0.32463 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19258 \text{ kip/ft}^2)}{(0.32463 \text{ kip/ft}^2)}$$

$$Ratio = 0.59322$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.86391 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.92151$$

Status: **PASS**
Ratio: **0.590**

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

Considering z-direction:

$H_o = 0.067675 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.15924 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15924 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.067675 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.15924 \text{ kipft/ft})) + (4 \times (0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4995 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.15924 \text{ kipft/ft})) + (3 \times (0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.15924 \text{ kipft/ft})) + (2 \times (0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.052687 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.15924 \text{ kipft/ft})) + ((0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.11389 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4995 \text{ ft})}{2}$$

$$p_a = 0.33746 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.052687 \text{ kip/ft}^2)}{(0.33746 \text{ kip/ft}^2)}$$

$$Ratio = 0.15613$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

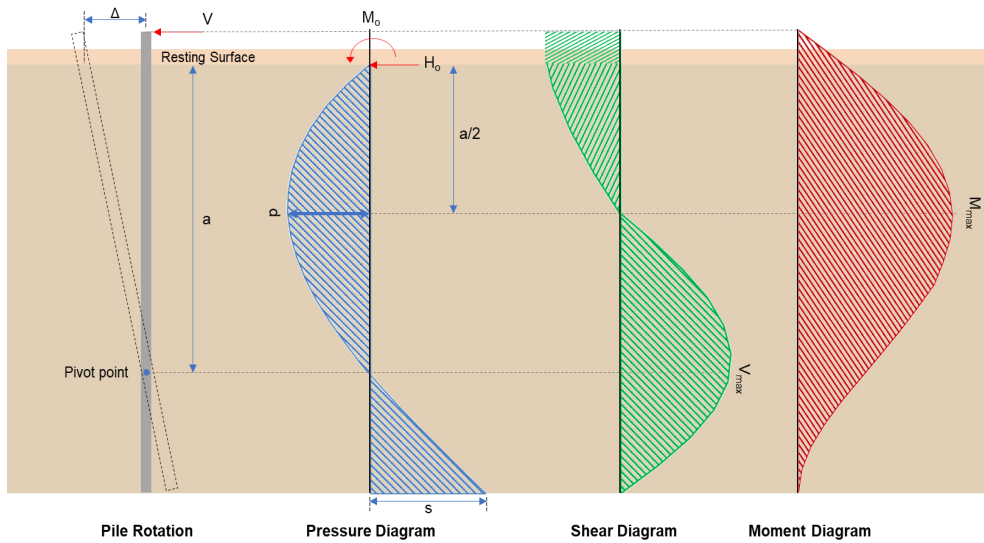
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.11389 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.12148$$

Status: **PASS**
Ratio: **0.160**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.825 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.76831 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.016 \text{ kipft}) + ((-4.825 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.1682 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.1682 \text{ kipft/ft})}{(-0.76831 \text{ kip/ft})}$$

$$E = 9.3297 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.1682 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.1682 \text{ kipft/ft})) + (4 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.76831 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (7.1682 \text{ kipft/ft})) + (4 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3275 \text{ ft}$$

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

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

$$V_{max} = ((-0.76831 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.3297 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3275 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.3297 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3275 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.144 \text{ kip}$$

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

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

$$M_{max} = ((-0.76831 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(9.3297 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.3297 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.3297 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 29.96 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.648 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10318 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.551 \text{ kipft}) + ((0.648 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.24697 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.24697 \text{ kipft/ft})}{(0.10318 \text{ kip/ft})}$$

$$E = 2.3935 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.24697 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.10318 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.24697 \text{ kipft/ft})) + (4 \times (0.10318 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4975 \text{ ft}$$

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

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

$$V_{max} = ((0.10318 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.3935 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4975 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.3935 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4975 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.55582 \text{ kip}$$

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

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

$$M_{max} = ((0.10318 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.3935 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4975 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.3935 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4975 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.3935 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4975 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5077 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.433 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.249 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.249 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Min[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Min[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.433 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0039$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.433 \text{ kip} \rightarrow 10433 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10433 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.88 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.88 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.88 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((119.88 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111 \text{ kip}$$

Considering x-direction:

V_{max} = 10.144 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(10.144 \text{ kip})}{(111 \text{ kip})}$$

$$Ratio = 0.091388$$

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

Considering z-direction:

$V_{max} = 0.55582 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.55582 \text{ kip})}{(111 \text{ kip})}$$

$$Ratio = 0.0050074$$

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

Flexural Strength (ACI 318-19, LFRD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

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:

$M_{max} = 29.96 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(29.96 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.12003$$

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

Considering z-direction:

$M_{max} = 1.5077 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.5077 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0060403$$

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

REFERENCES	CALCULATIONS	RESULTS
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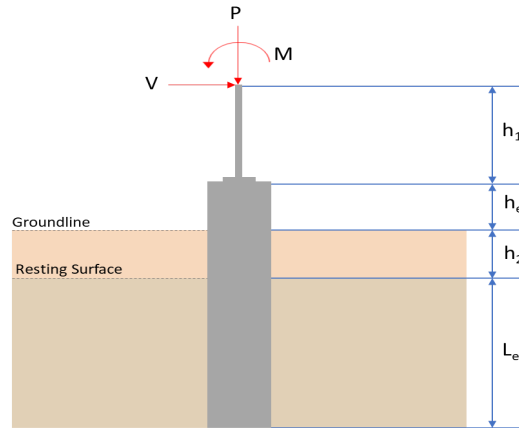
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

$h_1 = 0$ ft - Lateral load height from the top of the pile,

$h_2 = 0$ ft - Depth to resisting surface

$h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	6.664	10.433
V_x (kip)	-2.882	-4.825
V_z (kip)	-0.425	-0.648
M_x (kipft)	-1.000	-1.554
M_z (kipft)	26.667	45.017

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.882 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.45892 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(26.667 \text{ kipft}) + ((-2.882 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.2463 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6805 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.425 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.067675 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1 \text{ kipft}) + ((-0.425 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.15924 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7705 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6805 \text{ ft}), (1.7705 \text{ ft})]$$

$$L_{e,req} = 5.68 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.68 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9088$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.664 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.4165 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.4165 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.20825$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.45892 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 4.2463 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.2463 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (4.2463 \text{ kipft/ft})) + (4 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3284 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.2463 \text{ kipft/ft})) + (3 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (4.2463 \text{ kipft/ft})) + (2 \times (-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.19258 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (4.2463 \text{ kipft/ft})) + ((-0.45892 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.86391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3284 \text{ ft})}{2}$$

$$p_a = 0.32463 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.19258 \text{ kip/ft}^2)}{(0.32463 \text{ kip/ft}^2)}$$

$$Ratio = 0.59322$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.86391 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.92151$$

Status: **PASS**
Ratio: **0.590**

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

Considering z-direction:

$H_o = -0.067675 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.15924 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.15924 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.067675 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.15924 \text{ kipft/ft})) + (4 \times (-0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4995 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.15924 \text{ kipft/ft})) + (3 \times (-0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.15924 \text{ kipft/ft})) + (2 \times (-0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.020824 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.15924 \text{ kipft/ft})) + ((-0.067675 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.016051 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4995 \text{ ft})}{2}$$

$$p_a = 0.33746 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.020824 \text{ kip/ft}^2)}{(0.33746 \text{ kip/ft}^2)}$$

$$Ratio = -0.061708$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 0.9375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

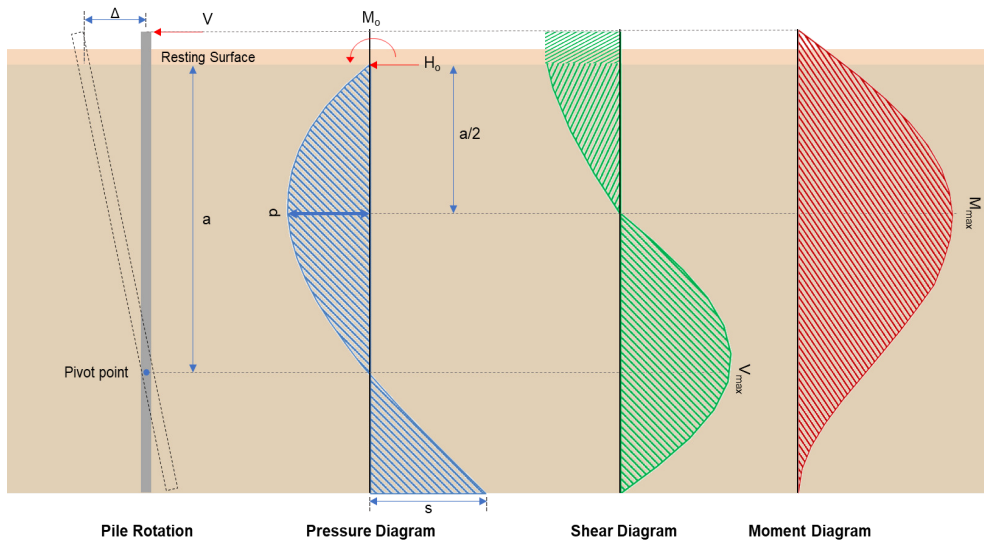
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.016051 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = -0.017121$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.825 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.76831 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(45.017 \text{ kipft}) + ((-4.825 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.1683 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.1683 \text{ kipft/ft})}{(-0.76831 \text{ kip/ft})}$$

$$E = 9.33 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.1683 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (7.1683 \text{ kipft/ft})) + (4 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{6 \times (7.1683 \text{ kipft/ft}) + (4 \times (-0.76831 \text{ kip/ft}) \times (6.25 \text{ ft}))}{\dots}$$

$$a = 4.3275 \text{ ft}$$

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

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

$$V_{max} = ((-0.76831 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.33 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3275 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.33 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3275 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 10.144 \text{ kip}$$

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

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

$$M_{max} = ((-0.76831 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(9.33 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.33 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.33 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3275 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 29.961 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.648 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10318 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.554 \text{ kipft}) + ((-0.648 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.24745 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.24745 \text{ kipft/ft})}{(-0.10318 \text{ kip/ft})}$$

$$E = 2.3981 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.24745 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.10318 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.24745 \text{ kipft/ft})) + (4 \times (-0.10318 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4972 \text{ ft}$$

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

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

$$V_{max} = ((-0.10318 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.3981 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4972 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (2.3981 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4972 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.55636 \text{ kip}$$

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

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

$$M_{max} = ((-0.10318 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.3981 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4972 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (2.3981 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4972 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.3981 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4972 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.5093 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(10.433 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.249 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.249 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Min[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Min[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(10.433 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0039$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</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>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 10.433 \text{ kip} \rightarrow 10433 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(10433 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.88 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.88 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.88 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((119.88 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111 \text{ kip}$$

Considering x-direction:

V_{max} = 10.144 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(10.144 \text{ kip})}{(111 \text{ kip})}$$

$$Ratio = 0.09139$$

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

Considering z-direction:

$V_{max} = 0.55636 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.55636 \text{ kip})}{(111 \text{ kip})}$$

$$Ratio = 0.0050122$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

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:

$M_{max} = 29.961 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(29.961 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.12003$$

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

Considering z-direction:

$M_{max} = 1.5093 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.5093 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0060469$$

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

REFERENCES	CALCULATIONS	RESULTS
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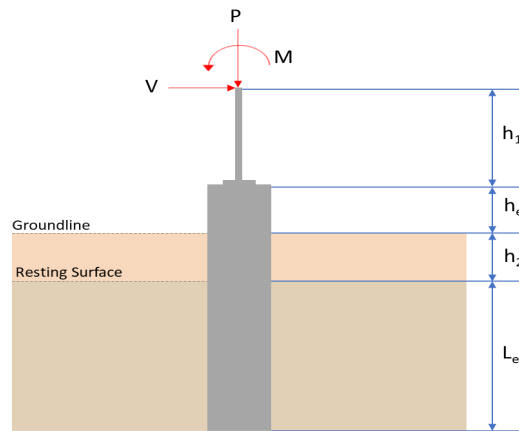
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.75$ 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

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.624	13.515
V_x (kip)	-3.806	-6.362
V_z (kip)	-0.017	-0.029
M_x (kipft)	-0.044	-0.069
M_z (kipft)	35.427	59.974

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.806 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.427 \text{ kipft}) + ((-3.806 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6412 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1169 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.002707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.044 \text{ kipft}) + ((-0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0070064 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.75907 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.1169 \text{ ft}), (0.75907 \text{ ft})]$$

$$L_{e,req} = 6.117 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.117 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.90622$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.624 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.539 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.539 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2695$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.60605 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6412 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6412 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.6412 \text{ kipft/ft})) + (4 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6833 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6412 \text{ kipft/ft})) + (3 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (5.6412 \text{ kipft/ft})) + (2 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.19947 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6412 \text{ kipft/ft})) + ((-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.94705 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6833 \text{ ft})}{2}$$

$$p_a = 0.35125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19947 \text{ kip/ft}^2)}{(0.35125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56789$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.94705 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93536$$

Status: **PASS**
Ratio: **0.570**

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

Considering z-direction:

$H_o = -0.002707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0070064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0070064 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0070064 \text{ kipft/ft})) + (4 \times (-0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8571 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0070064 \text{ kipft/ft})) + (3 \times (-0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.0070064 \text{ kipft/ft})) + (2 \times (-0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = -0.00076103 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0070064 \text{ kipft/ft})) + ((-0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = -0.00056093 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8571 \text{ ft})}{2}$$

$$p_a = 0.36428 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00076103 \text{ kip/ft}^2)}{(0.36428 \text{ kip/ft}^2)}$$

$$Ratio = -0.0020891$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

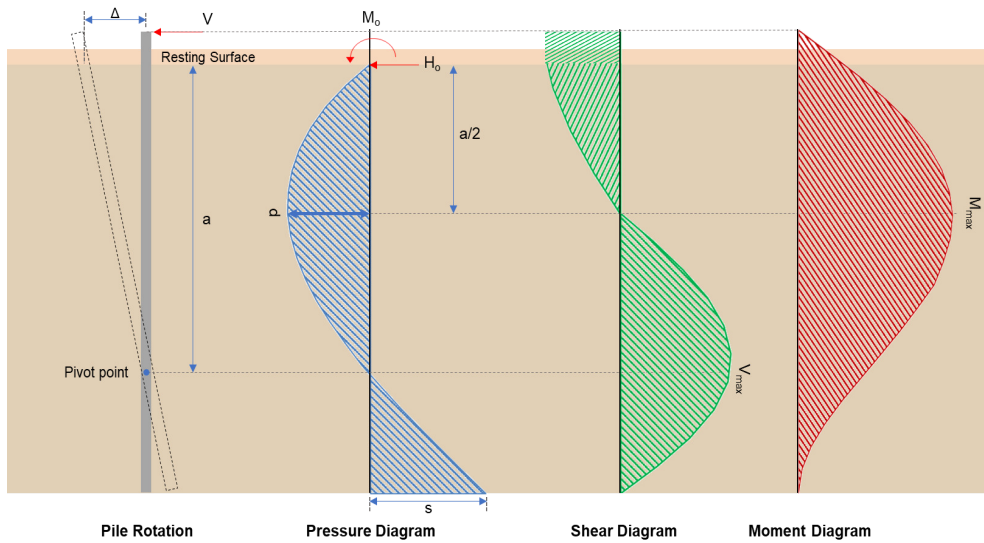
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.00056093 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = -0.000554$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.362 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0131 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(59.974 \text{ kipft}) + ((-6.362 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.55 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.55 \text{ kipft/ft})}{(-1.0131 \text{ kip/ft})}$$

$$E = 9.4269 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.55 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{6 \times (9.55 \text{ kipft/ft}) + 4 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft})}$$

$$a = \frac{(6 \times (9.55 \text{ kipft/ft})) + (4 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft}))}{\dots}$$

$$a = 4.6818 \text{ ft}$$

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

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

$$V_{max} = ((-1.0131 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6818 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6818 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.686 \text{ kip}$$

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

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

$$M_{max} = ((-1.0131 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(9.4269 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 40.339 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.029 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.069 \text{ kipft}) + ((-0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010987 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010987 \text{ kipft/ft})}{(-0.0046178 \text{ kip/ft})}$$

$$E = 2.3793 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010987 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.0046178 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.010987 \text{ kipft/ft})) + (4 \times (-0.0046178 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.868 \text{ ft}$$

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

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

$$V_{max} = ((-0.0046178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.868 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.868 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023895 \text{ kip}$$

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

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

$$M_{max} = ((-0.0046178 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(2.3793 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.069573 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.515 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.147 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.147 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<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><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.515 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.005052$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.515 \text{ kip} \rightarrow 13515 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13515 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.29 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.29 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.29 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((120.29 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.27 \text{ kip}$$

Considering x-direction:

V_{max} = 12.686 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(12.686 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.11401$$

Considering z-direction:

$V_{max} = 0.023895 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.023895 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.00021475$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$$

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

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

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:

$M_{max} = 40.339 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(40.339 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.16161$$

Status: **PASS**
Ratio: **0.160**

Considering z-direction:

$M_{max} = 0.069573 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.069573 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00027874$$

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

REFERENCES	CALCULATIONS	RESULTS
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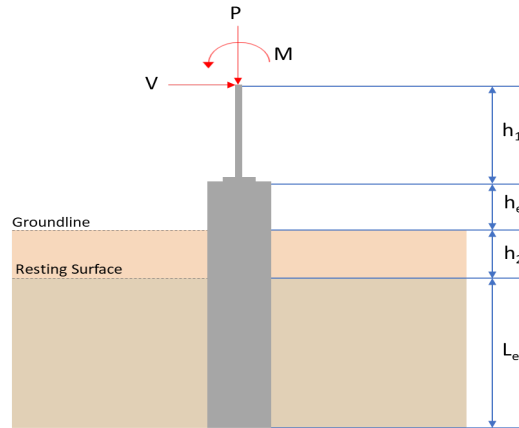
SkyCiv Foundation Design

Pile Foundation

Design Information :

Design code : IBC 2021 (International Building Code)
Unit System : Imperial

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.75$ 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

Tabulation of Soil Parameters

Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)
1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.624	13.515
V_x (kip)	-3.806	-6.362
V_z (kip)	0.017	0.029
M_x (kipft)	0.044	0.069
M_z (kipft)	35.427	59.974

Material Properties

$f'_{ck} = 2.5$ ksi - Concrete strength.

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

$$H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$$

$$H = 0 \text{ ft}$$

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.806 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.60605 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.427 \text{ kipft}) + ((-3.806 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6412 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1169 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.017 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.002707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.044 \text{ kipft}) + ((0.017 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0070064 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.89025 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.1169 \text{ ft}), (0.89025 \text{ ft})]$$

$$L_{e,req} = 6.117 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.117 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.90622$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.624 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.539 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.539 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.2695$$

Status: **PASS**
Ratio: **0.270**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.60605 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.6412 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6412 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (5.6412 \text{ kipft/ft})) + (4 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6833 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (5.6412 \text{ kipft/ft})) + (3 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (5.6412 \text{ kipft/ft})) + (2 \times (-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.19947 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (5.6412 \text{ kipft/ft})) + ((-0.60605 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.94705 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6833 \text{ ft})}{2}$$

$$p_a = 0.35125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.19947 \text{ kip/ft}^2)}{(0.35125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.56789$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.94705 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.93536$$

Status: **PASS**
Ratio: **0.570**

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

Considering z-direction:

$H_o = 0.002707 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0070064 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0070064 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.0070064 \text{ kipft/ft})) + (4 \times (0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.8571 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.0070064 \text{ kipft/ft})) + (3 \times (0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 [(3 \times (0.0070064 \text{ kipft/ft})) + (2 \times (0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.0019625 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0070064 \text{ kipft/ft})) + ((0.002707 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.0042515 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.8571 \text{ ft})}{2}$$

$$p_a = 0.36428 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0019625 \text{ kip/ft}^2)}{(0.36428 \text{ kip/ft}^2)}$$

$$Ratio = 0.0053873$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

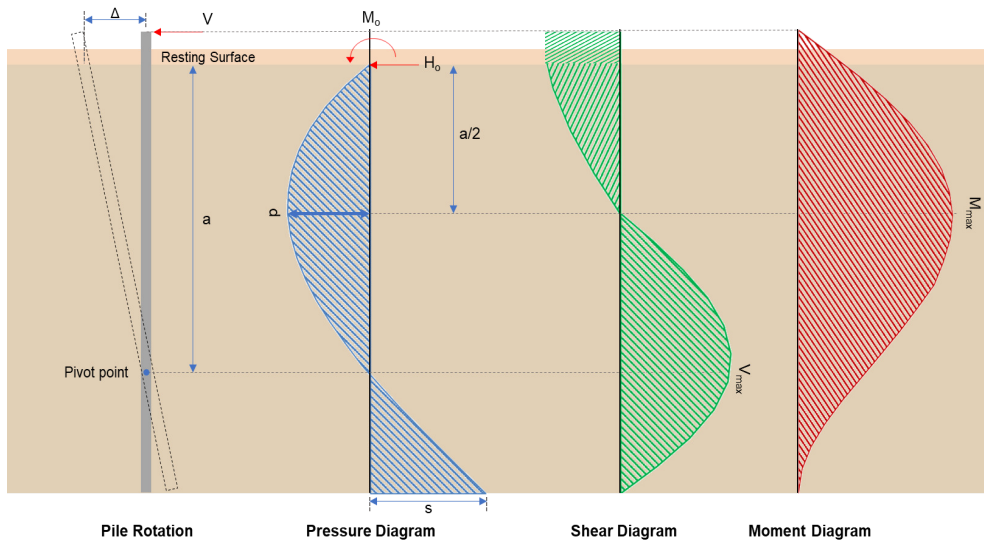
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0042515 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.004199$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-6.362 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0131 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(59.974 \text{ kipft}) + ((-6.362 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 9.55 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(9.55 \text{ kipft/ft})}{(-1.0131 \text{ kip/ft})}$$

$$E = 9.4269 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (9.55 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (9.55 \text{ kipft/ft})) + (4 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = \frac{(6 \times (9.55 \text{ kipft/ft})) + (4 \times (-1.0131 \text{ kip/ft}) \times (6.75 \text{ ft}))}{}$$

$$a = 4.6818 \text{ ft}$$

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

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

$$V_{max} = ((-1.0131 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6818 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6818 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 12.686 \text{ kip}$$

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

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

$$M_{max} = ((-1.0131 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(9.4269 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (9.4269 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.6818 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 40.339 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.029 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0046178 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.069 \text{ kipft}) + ((0.029 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010987 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010987 \text{ kipft/ft})}{(0.0046178 \text{ kip/ft})}$$

$$E = 2.3793 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010987 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.0046178 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.010987 \text{ kipft/ft})) + (4 \times (0.0046178 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.868 \text{ ft}$$

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

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

$$V_{max} = ((0.0046178 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.868 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.868 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.023895 \text{ kip}$$

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

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

$$M_{max} = ((0.0046178 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(2.3793 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.3793 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.868 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.069573 \text{ kipft}$$

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,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.515 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.147 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.147 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.515 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.005052$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<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> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.515 \text{ kip} \rightarrow 13515 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13515 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.29 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.29 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.29 \text{ kip}$$

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

$$V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$$

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 737.28 \text{ kip}$$

A_v - Ties rebar area,

$$A_v = \frac{\pi d_{ties}^2}{4}$$

$$A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$$

$$A_v = 0.11045 \text{ in}^2$$

22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)

$$V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$$

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 50.894 \text{ kip}$$

V_s - Governing shear strength of steel

$$V_s = \text{MIN}[V_{s,a}, V_{s,b}]$$

$$V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$$

$$V_s = 50.894 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

$$\phi V_n = \phi (V_c + V_s)$$

$$\phi V_n = (0.65) \times ((120.29 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.27 \text{ kip}$$

Considering x-direction:

V_{max} = 12.686 kip - Maximum shear force in the x-direction,

Ratio - Capacity

$$\text{Ratio} = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(12.686 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.11401$$

Considering z-direction:

$V_{max} = 0.023895 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

$$Ratio = \frac{V_{max}}{\phi V_n}$$

$$Ratio = \frac{(0.023895 \text{ kip})}{(111.27 \text{ kip})}$$

$$Ratio = 0.00021475$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{b D^2}{6}$$

$$S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$$

$$S_m = 18432 \text{ in}^3$$

$\lambda = 1$ - Concrete modification factor (Normal concrete),

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

$$\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$$

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 18432.001 \text{ in}^3$$

$$\phi M_{n,1} = 249.600 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

$$\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$$

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

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

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:

$M_{max} = 40.339 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(40.339 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.16161$$

Status: **PASS**
Ratio: **0.160**

Considering z-direction:

$M_{max} = 0.069573 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.069573 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00027874$$

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