

Project Name: Giesey ExpC 75deg 5x12 clay -

V1Jb

Date: Wed Sep 24 2025

Location: 8385 Jordan Rd, Melvin, MI 48454, USA

Number of Modules: 60

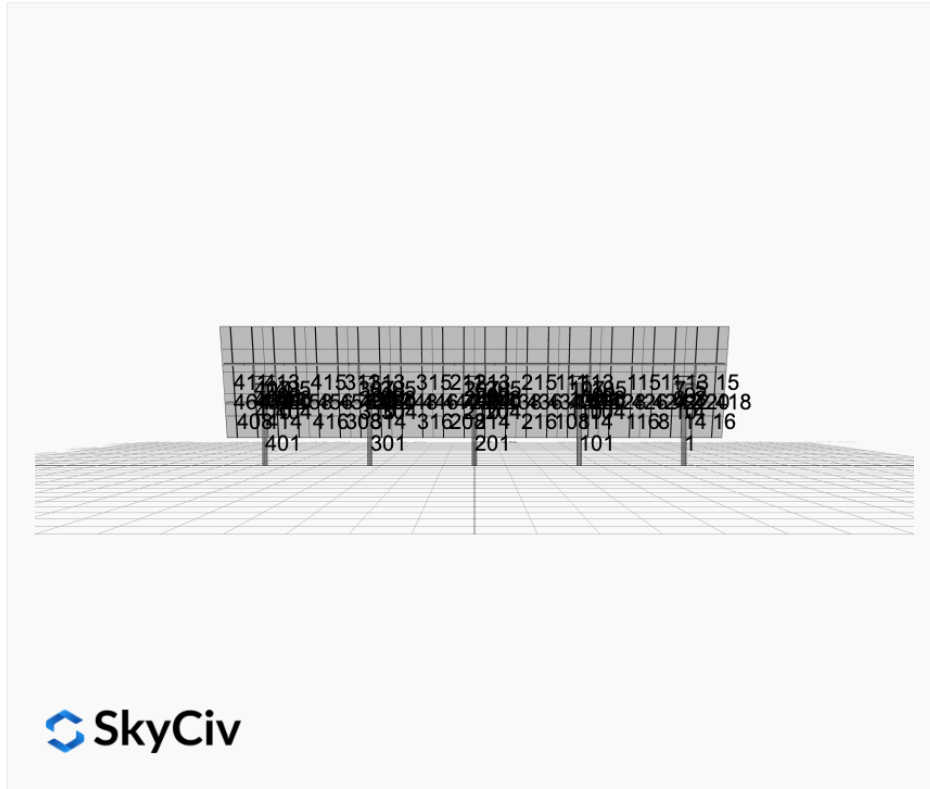
Unique ID: 5P-19.75-10TOP-HD-45-L-5Hx12W-

Number of Poles: 5

C26C

Date Sold: _____

Dealer: _____



Array Dimensions N/S	21.58 ft
Array Dimensions E/W	94.90 ft
Winter Tilt Angle	75
Front Edge Clearance	5 ft

MT Solar Bill of Materials (5P-19.75-10TOP-HD-45-L-5Hx12W-C26C)

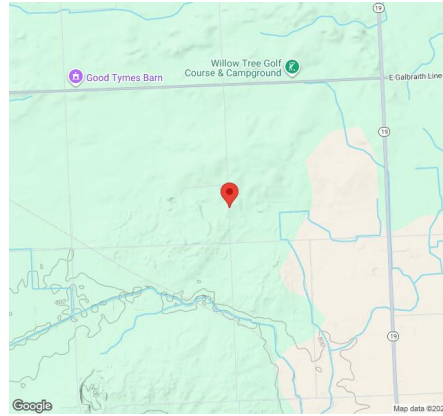
Part	Short Description	BOM Qty
MTS-PC-10	10IN Pole Cap Assembly	5
MTS-HF-HD	H-Frame Assembly-HD	5
MTS-HD-Wing-45	45IN HD Wing	4
MTS-HD-Splice-90	90IN HD Splice	8
MTS-HD-Splice-57	57IN HD Splice	8
MTS-CLAMP-ANGLE-4PK	Angle Clamp	12

Rail Bill of Materials

Part	Qty
Rails (259in)	24
Rail Attachment	96
Module Mid Clamp	96
Module End Clamp	48

Part	Qty
Ground Lug	12

Site Details:



Site Address: 8385 Jordan Rd, Melvin, MI 48454, USA

Array Specification

Duty Classification:	HD
Module Width:	51.30 in
Module Length:	93.90in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Winter Tilt Angle:	75
Front Edge Clearance:	5
Total Array Height at Tilt:	25.85 ft
Total Frame Length:	94.00 ft
Module Info/Notes:	Risen RSM132-8-720-740BHDG Thornova Solar TS-BGT66(700-720)-G12
Array Dimensions N/S:	21.58 ft
Array Dimensions E/W:	94.90 ft
Rail Length:	259.00 in
Rail Spacing:	3.95 ft

Support Specifications

Pole Size:	10in Pipe Sch 80
Pole Length above Grade:	15.42 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 11.00 ft Pile 2: 11.25 ft Pile 3: 11.25 ft Pile 4: 11.25 ft Pile 5: 11.00 ft
Foundation Volume:	33.037 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	clay
Site Location:	8385 Jordan Rd, Melvin, MI 48454, USA

Wind Speed:	101 mph
Snow Load:	25 psf

Design Disclaimer

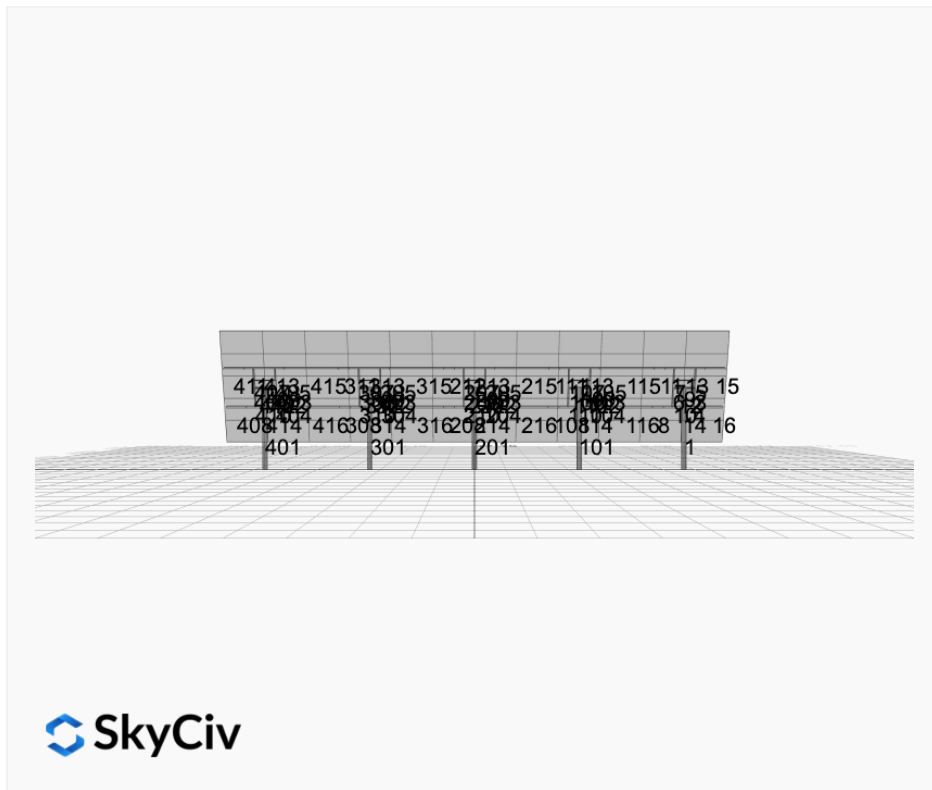
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

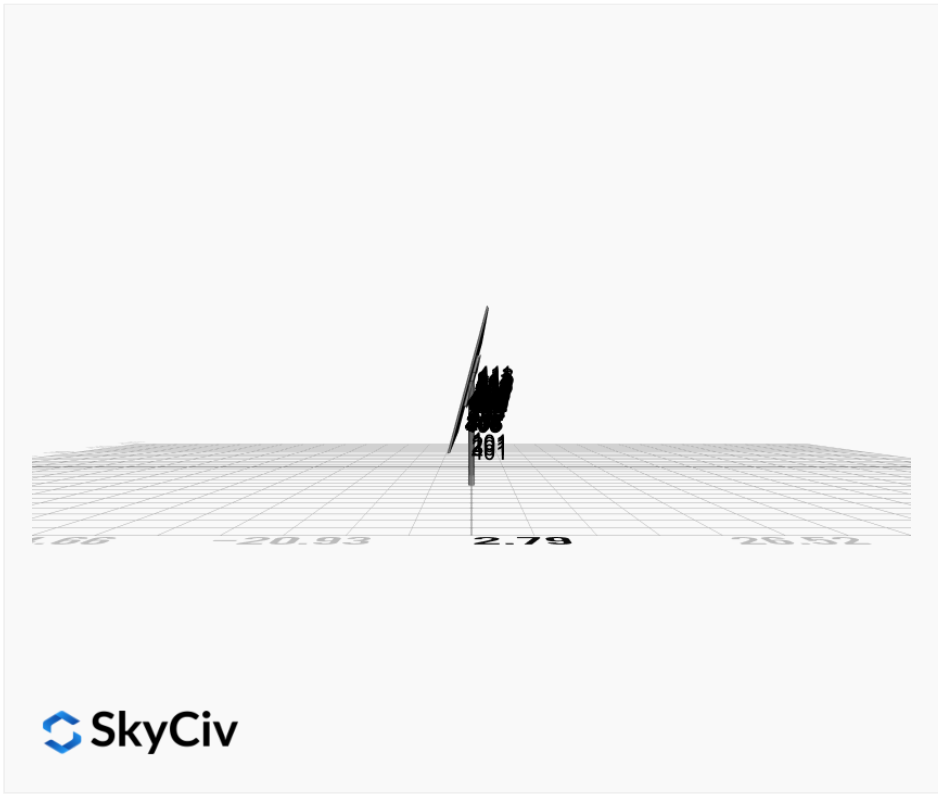
AutoDesigner Input

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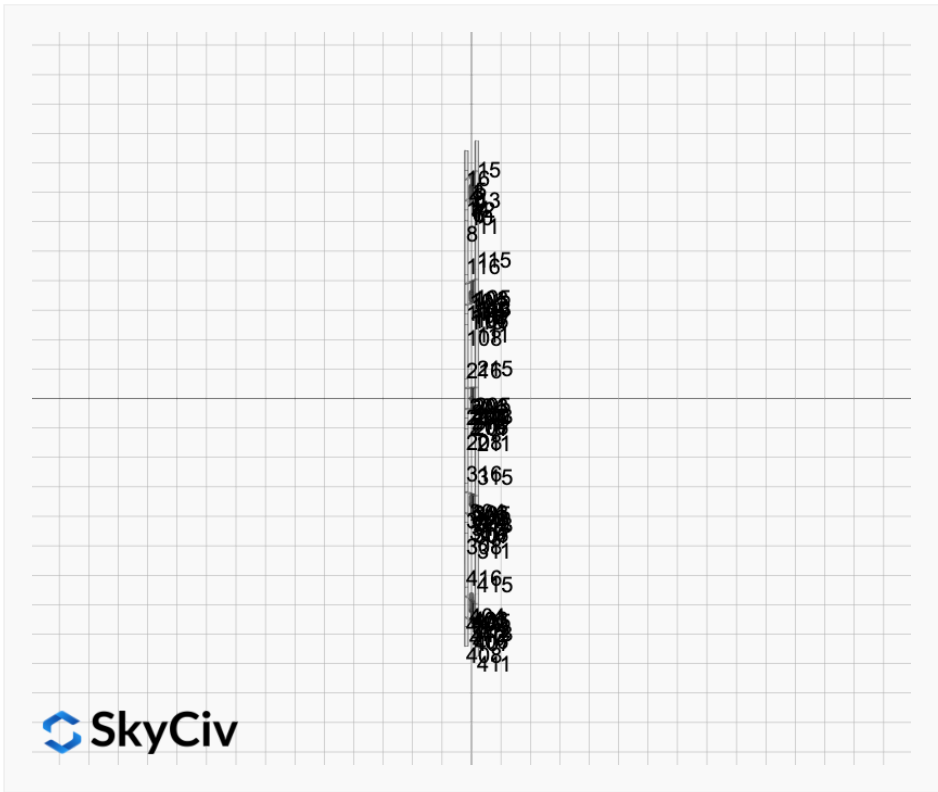
Design Notes:

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





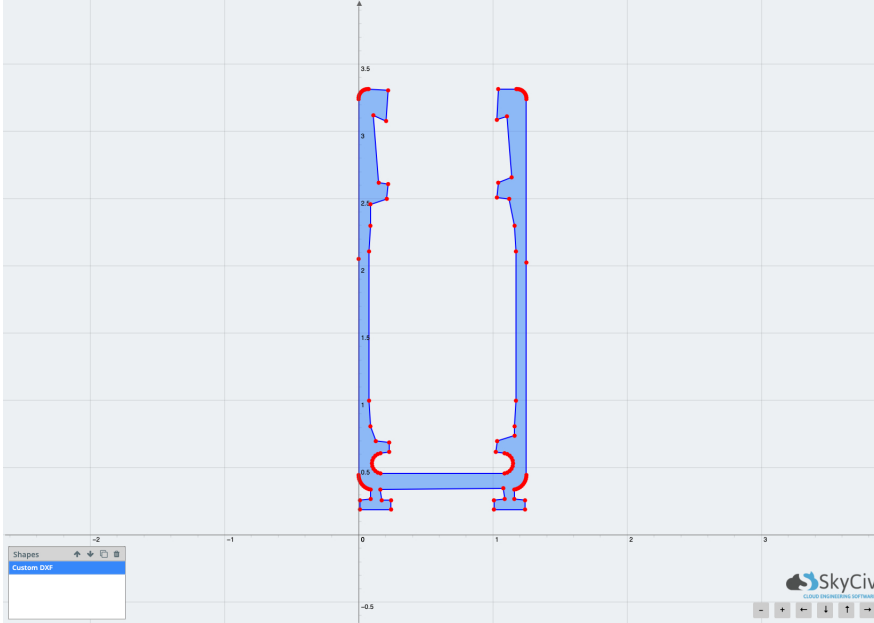
 SkyCiv



 SkyCiv

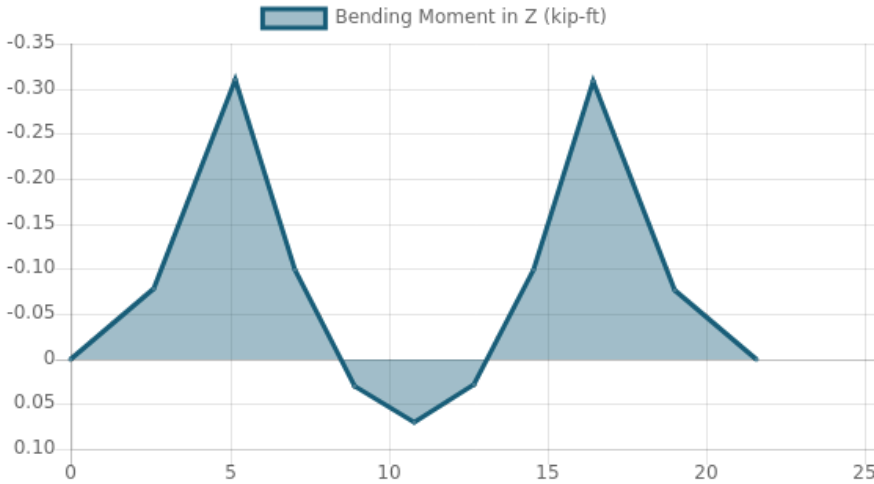
Rail Design Check

Rail Length: 21.583333333333332 ft
Additional Restraints Required: 4ft Spread Clamps
Tributary Width: 3.954166666666667 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Wind uplift Case A (X): 0.0000 kip/ft
Wind uplift Case A (Y): 0.1161 kip/ft
Wind downforce Case A: -0.1161 kip/ft
Dead (Panel load): -0.1161 kip/ft

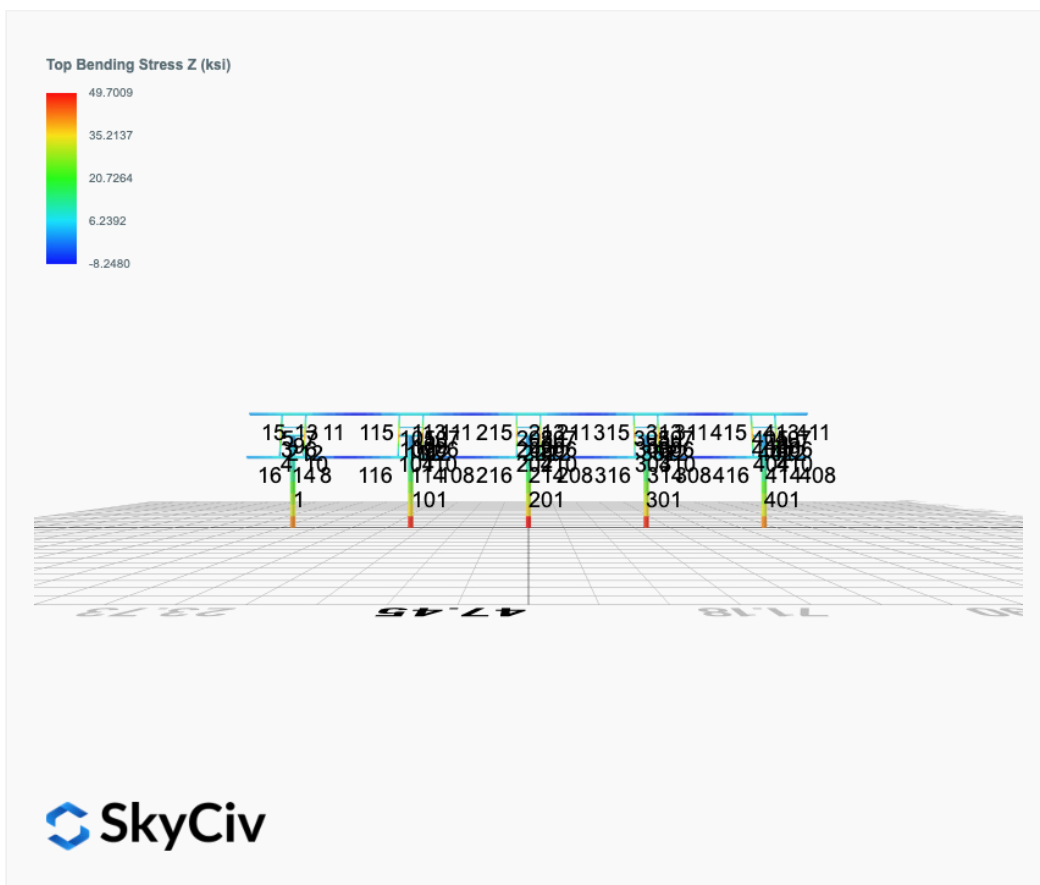
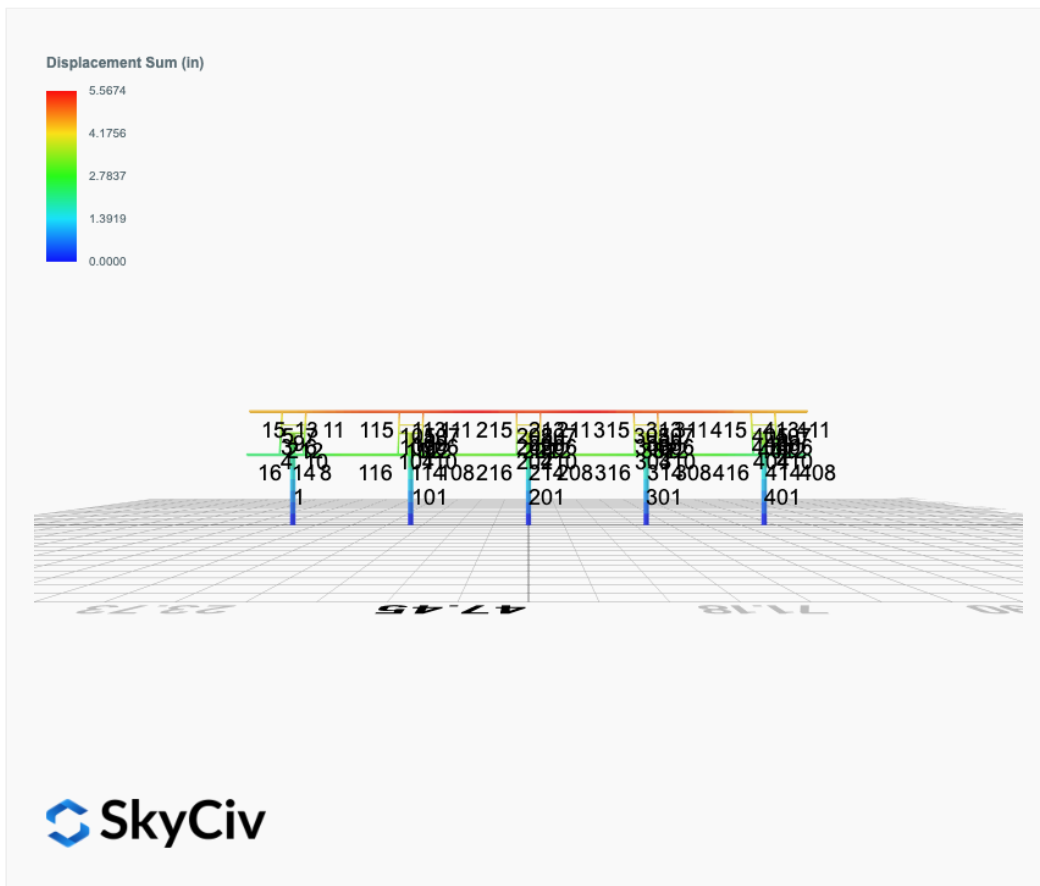


Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	39.17615015	1.136	FAIL
Material Yield	34.5	39.17615015	1.136	FAIL
Material Strength	37	39.17615015	1.059	FAIL

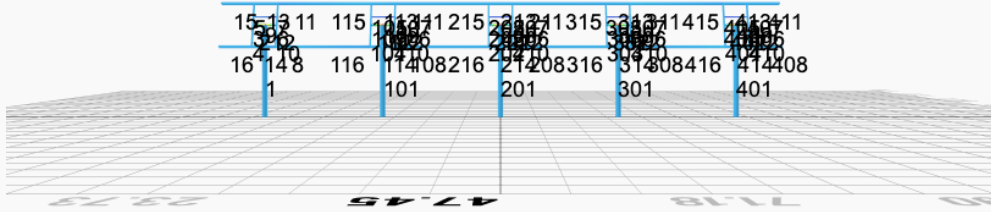
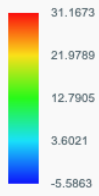
Member 1, ULS: 1. 1.4D



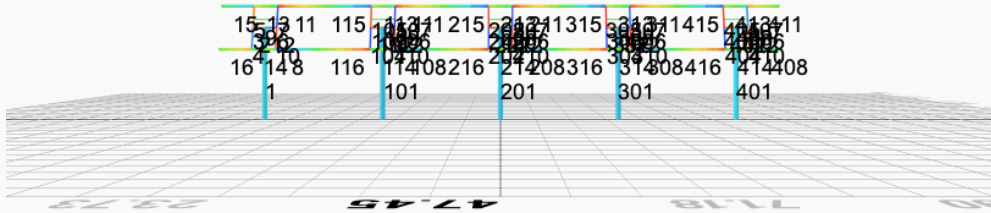
FEM Results (Envelope Worst Case for each member)



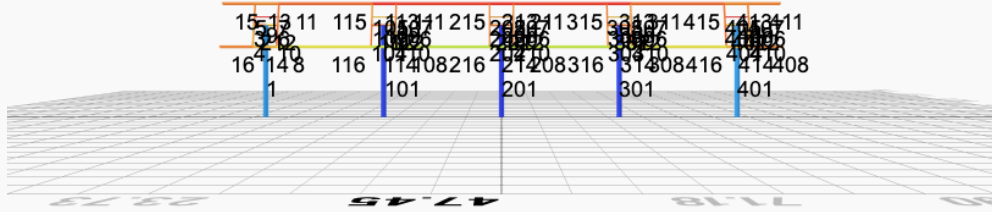
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 2. D + L	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 3. D + (S or Lr or R)	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 3. D + (S or Lr or R)	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 5b. D + 0.7E	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 8. 0.6D + 0.7E	0.0026	1.9206	0.0180	0.0833	-0.0031	-0.0334
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.4600	4.9220	0.0826	0.3471	-1.5817	100.2535
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.4669	1.4804	-0.0220	-0.0669	1.5551	-99.4128
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.8439	4.4918	0.0694	0.2951	-1.1876	75.1762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8512	1.9106	-0.0090	-0.0155	1.1650	-74.5735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.8439	4.4918	0.0694	0.2951	-1.1876	75.1762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8512	1.9106	-0.0090	-0.0155	1.1650	-74.5735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0043	3.2011	0.0300	0.1389	-0.0052	-0.0557
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.4617	3.6416	0.0706	0.2916	-1.5796	100.2758
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0026	1.9206	0.0180	0.0833	-0.0031	-0.0334
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.4651	0.2000	-0.0340	-0.1225	1.5572	-99.3905
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0026	1.9206	0.0180	0.0833	-0.0031	-0.0334

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	6.7098
Shear X	-10.7776
Shear Z	0.1241
Moment X	0.5155
Moment Y (Twist)	2.6448
Moment Z	168.2631

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	4.9220
Shear X	-6.4669
Shear Z	0.0826
Moment X	0.3471
Moment Y (Twist)	1.5817
Moment Z	100.2758

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	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 2. D + L	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 3. D + (S or Lr or R)	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 3. D + (S or Lr or R)	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 5b. D + 0.7E	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 8. 0.6D + 0.7E	-0.0022	2.0860	-0.0000	-0.0006	0.0018	0.0389
ULS: 5a. D + 0.6W_Wind downforce Case A only	-7.2861	5.4376	0.0119	0.0442	-0.4214	112.8211
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 5a. D + 0.6W_Wind uplift Case A only	7.2805	1.5153	-0.0116	-0.0446	0.4147	-111.5441
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.4655	4.9473	0.0089	0.0329	-0.3153	84.6321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.4594	2.0056	-0.0087	-0.0337	0.3118	-83.6418
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.4655	4.9473	0.0089	0.0329	-0.3153	84.6321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.4594	2.0056	-0.0087	-0.0337	0.3118	-83.6418
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0037	3.4766	-0.0001	-0.0010	0.0029	0.0648
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-7.2846	4.0469	0.0119	0.0446	-0.4226	112.7952
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0022	2.0860	-0.0000	-0.0006	0.0018	0.0389
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	7.2819	0.1247	-0.0116	-0.0442	0.4136	-111.5700
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0022	2.0860	-0.0000	-0.0006	0.0018	0.0389

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	7.4398
Shear X	-12.1403
Shear Z	0.0202
Moment X	0.0754
Moment Y (Twist)	0.7154
Moment Z	189.3464

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.4376
Shear X	-7.2861
Shear Z	0.0119
Moment X	-0.0446
Moment Y (Twist)	0.4226
Moment Z	112.8211

Reaction Forces for Foundation 3 (Node ID#201), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 2. D + L	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 3. D + (S or Lr or R)	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 3. D + (S or Lr or R)	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 5b. D + 0.7E	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 8. 0.6D + 0.7E	-0.0007	2.0854	0.0000	-0.0000	0.0000	0.0178
ULS: 5a. D + 0.6W_Wind downforce Case A only	-7.3769	5.4551	0.0000	-0.0000	0.0000	114.4382
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 5a. D + 0.6W_Wind uplift Case A only	7.3745	1.4965	0.0000	-0.0000	0.0000	-113.1699
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297

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	-5.5330	4.9603	0.0000	-0.0000	0.0000	85.8361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5306	1.9913	0.0000	-0.0000	0.0000	-84.8700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.5330	4.9603	0.0000	-0.0000	0.0000	85.8361
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.5306	1.9913	0.0000	-0.0000	0.0000	-84.8700
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0011	3.4757	0.0000	-0.0000	0.0000	0.0297
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-7.3764	4.0648	0.0000	-0.0000	0.0000	114.4264
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0007	2.0854	0.0000	-0.0000	0.0000	0.0178
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	7.3749	0.1062	0.0000	-0.0000	0.0000	-113.1817
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0007	2.0854	0.0000	-0.0000	0.0000	0.0178

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	7.4700
Shear X	-12.2947
Shear Z	0.0000
Moment X	-0.0001
Moment Y (Twist)	0.0002
Moment Z	192.1167

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	5.4551
Shear X	-7.3769
Shear Z	0.0000
Moment X	-0.0000
Moment Y (Twist)	0.0000
Moment Z	114.4382

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	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 2. D + L	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 3. D + (S or Lr or R)	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 3. D + (S or Lr or R)	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 5b. D + 0.7E	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 8. 0.6D + 0.7E	-0.0022	2.0860	0.0000	0.0006	-0.0017	0.0389
ULS: 5a. D + 0.6W_Wind downforce Case A only	-7.2861	5.4376	-0.0119	-0.0442	0.4214	112.8211
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 5a. D + 0.6W_Wind uplift Case A only	7.2805	1.5153	0.0116	0.0446	-0.4147	-111.5441
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.4655	4.9473	-0.0089	-0.0329	0.3153	84.6321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.4594	2.0056	0.0087	0.0337	-0.3118	-83.6418
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-5.4655	4.9473	-0.0089	-0.0329	0.3153	84.6321
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	5.4594	2.0056	0.0087	0.0337	-0.3118	-83.6418
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0037	3.4766	0.0001	0.0010	-0.0029	0.0648

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-7.2846	4.0469	-0.0119	-0.0446	0.4226	112.7952
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0022	2.0860	0.0000	0.0006	-0.0017	0.0389
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	7.2819	0.1247	0.0116	0.0442	-0.4136	-111.5700
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0022	2.0860	0.0000	0.0006	-0.0017	0.0389

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	7.4398
Shear X	-12.1403
Shear Z	-0.0202
Moment X	-0.0755
Moment Y (Twist)	0.7157
Moment Z	189.3465

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	5.4376
Shear X	-7.2861
Shear Z	-0.0119
Moment X	0.0446
Moment Y (Twist)	0.4226
Moment Z	112.8211

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 2. D + L	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 3. D + (S or Lr or R)	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 3. D + (S or Lr or R)	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 5b. D + 0.7E	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 8. 0.6D + 0.7E	0.0026	1.9206	-0.0180	-0.0834	0.0032	-0.0334
ULS: 5a. D + 0.6W_Wind downforce Case A only	-6.4600	4.9220	-0.0826	-0.3472	1.5817	100.2535
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 5a. D + 0.6W_Wind uplift Case A only	6.4669	1.4804	0.0220	0.0669	-1.5551	-99.4127
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.8439	4.4918	-0.0694	-0.2951	1.1876	75.1762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8512	1.9106	0.0090	0.0154	-1.1650	-74.5735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-4.8439	4.4918	-0.0694	-0.2951	1.1876	75.1762
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	4.8512	1.9106	0.0090	0.0154	-1.1650	-74.5735
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0043	3.2011	-0.0300	-0.1389	0.0053	-0.0557
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-6.4617	3.6416	-0.0706	-0.2916	1.5796	100.2758
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0026	1.9206	-0.0180	-0.0834	0.0032	-0.0334
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	6.4651	0.2000	0.0340	0.1225	-1.5572	-99.3905
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0026	1.9206	-0.0180	-0.0834	0.0032	-0.0334

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.

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	6.7098
Shear X	-10.7776
Shear Z	-0.1240
Moment X	-0.5156
Moment Y (Twist)	2.6449
Moment Z	168.2645

Result	Value (kip, kip-ft)
Axial	4.9220
Shear X	-6.4669
Shear Z	-0.0826
Moment X	-0.3472
Moment Y (Twist)	1.5817
Moment Z	100.2758

Project Details

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

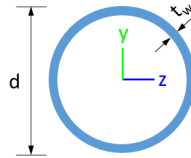


Design Input Information

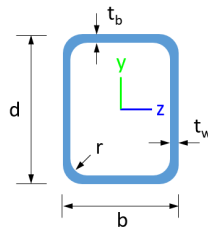
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

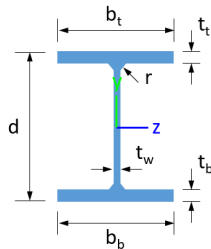
Section Dimensions



ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
12	10in Pipe Sch 80	10.75	0.59				



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



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

Section Properties

ID	Name	A (in ²)	J (in ⁴)	I _{yp} (in ⁴)	I _{zp} (in ⁴)	I _w (in ⁶)	S _{yp} (in ³)	S _{zp} (in ³)
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104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	123.95	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	123.95	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	85.85	23.93	6.12	40.24	43.62
114	133.20	85.85	23.89	6.12	40.24	43.62
115	133.20	69.16	17.31	6.12	40.24	43.62
116	133.20	69.16	17.58	6.12	40.24	43.62
201	851.50	362.61	229.67	229.67	255.45	255.45
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	123.95	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	123.95	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	85.85	23.76	6.12	40.24	43.62
214	133.20	85.85	23.74	6.12	40.24	43.62
215	133.20	69.16	17.79	6.12	40.24	43.62
216	133.20	69.16	17.87	6.12	40.24	43.62
301	851.50	362.61	229.67	229.67	255.45	255.45
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	123.95	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	123.95	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	23.93	6.12	40.24	43.62
314	133.20	85.85	23.89	6.12	40.24	43.62
315	133.20	69.16	17.57	6.12	40.24	43.62
316	133.20	69.16	17.71	6.12	40.24	43.62
401	851.50	362.61	229.67	229.67	255.45	255.45
402	198.33	196.72	21.95	21.95	59.50	59.50
403	116.10	115.41	15.79	11.10	42.08	23.28
404	116.10	111.33	15.79	11.10	42.08	23.28
405	116.10	114.23	15.79	11.10	42.08	23.28
406	116.10	115.41	15.79	11.10	42.08	23.28
407	116.10	114.23	15.79	11.10	42.08	23.28

407	110.10	114.23	13.79	11.10	42.00	23.20
408	133.20	52.83	32.87	6.12	40.24	43.62
409	66.48	58.89	3.82	3.82	19.94	19.94
410	116.10	111.33	15.79	11.10	42.08	23.28
411	133.20	52.83	32.87	6.12	40.24	43.62
412	198.33	196.72	21.95	21.95	59.50	59.50
413	133.20	85.85	24.69	6.12	40.24	43.62
414	133.20	85.85	24.81	6.12	40.24	43.62
415	133.20	69.16	17.44	6.12	40.24	43.62
416	133.20	69.16	17.41	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	0.019	0.733	0.006	0.042	0.000	0.744	#13	0.540	Not Required	Pass
2	0.002	0.159	0.384	0.041	0.082	0.544	#13	0.035	Not Required	Pass
3	0.006	0.640	0.030	0.063	0.004	0.668	#13	0.045	Not Required	Pass
4	0.005	0.640	0.080	0.064	0.017	0.698	#13	0.080	Not Required	Pass
5	0.005	0.398	0.081	0.064	0.020	0.415	#13	0.074	Not Required	Pass
6	0.007	0.778	0.050	0.079	0.010	0.825	#13	0.045	Not Required	Pass
7	0.007	0.484	0.109	0.077	0.027	0.507	#13	0.074	Not Required	Pass
8	0.002	0.088	0.096	0.051	0.011	0.126	#13	0.095	Not Required	Pass
9	0.007	0.036	0.098	0.002	0.002	0.121	#15	0.204	Not Required	Pass
10	0.007	0.758	0.104	0.076	0.022	0.812	#13	0.080	Not Required	Pass
11	0.001	0.081	0.098	0.053	0.011	0.119	#15	0.095	Not Required	Pass
12	0.002	0.223	0.505	0.052	0.100	0.728	#13	0.035	Not Required	Pass
13	0.004	0.237	0.253	0.067	0.014	0.417	#15	0.286	Not Required	Pass
14	0.005	0.232	0.251	0.066	0.014	0.398	#15	0.190	Not Required	Pass
15	0.000	0.072	0.089	0.031	0.007	0.149	#13	Not Required	Not Required	Pass
16	0.000	0.072	0.089	0.031	0.007	0.149	#13	Not Required	Not Required	Pass
101	0.021	0.824	0.001	0.048	0.000	0.835	#13	0.540	Not Required	Pass
102	0.002	0.210	0.483	0.052	0.100	0.694	#13	0.035	Not Required	Pass
103	0.007	0.772	0.045	0.076	0.007	0.814	#13	0.045	Not Required	Pass
104	0.007	0.785	0.106	0.078	0.023	0.855	#13	0.080	Not Required	Pass
105	0.007	0.480	0.110	0.077	0.027	0.505	#13	0.074	Not Required	Pass
106	0.007	0.811	0.045	0.081	0.007	0.852	#13	0.045	Not Required	Pass
107	0.007	0.505	0.110	0.081	0.027	0.529	#13	0.074	Not Required	Pass
108	0.002	0.071	0.096	0.051	0.011	0.117	#13	0.095	Not Required	Pass
109	0.008	0.026	0.085	0.001	0.001	0.109	#13	0.204	Not Required	Pass
110	0.007	0.810	0.106	0.081	0.023	0.873	#13	0.080	Not Required	Pass
111	0.001	0.078	0.098	0.051	0.011	0.111	#13	0.095	Not Required	Pass
112	0.002	0.221	0.515	0.053	0.104	0.737	#13	0.035	Not Required	Pass
113	0.004	0.223	0.254	0.065	0.014	0.421	#13	0.286	Not Required	Pass
114	0.006	0.245	0.252	0.067	0.014	0.435	#13	0.286	Not Required	Pass
115	0.002	0.325	0.141	0.050	0.011	0.446	#13	0.473	Not Required	Pass
116	0.002	0.318	0.141	0.052	0.011	0.440	#13	0.473	Not Required	Pass
201	0.021	0.836	0.000	0.048	0.000	0.847	#13	0.540	Not Required	Pass
202	0.002	0.217	0.507	0.053	0.103	0.725	#13	0.035	Not Required	Pass
203	0.007	0.807	0.045	0.080	0.007	0.848	#13	0.045	Not Required	Pass
204	0.007	0.801	0.106	0.080	0.023	0.867	#13	0.080	Not Required	Pass
205	0.007	0.502	0.110	0.080	0.027	0.526	#13	0.074	Not Required	Pass

206	0.007	0.807	0.045	0.080	0.007	0.848	#13	0.045	Not Required	Pass
207	0.007	0.502	0.110	0.080	0.027	0.526	#13	0.074	Not Required	Pass
208	0.002	0.067	0.096	0.052	0.011	0.126	#13	0.095	Not Required	Pass
209	0.008	0.026	0.082	0.001	0.000	0.102	#13	0.204	Not Required	Pass
210	0.007	0.801	0.106	0.080	0.023	0.867	#13	0.080	Not Required	Pass
211	0.001	0.071	0.098	0.052	0.011	0.123	#13	0.095	Not Required	Pass
212	0.002	0.217	0.507	0.053	0.103	0.725	#13	0.035	Not Required	Pass
213	0.004	0.251	0.253	0.067	0.014	0.448	#13	0.286	Not Required	Pass
214	0.006	0.261	0.251	0.067	0.014	0.451	#13	0.286	Not Required	Pass
215	0.003	0.295	0.141	0.052	0.011	0.416	#13	0.473	Not Required	Pass
216	0.002	0.283	0.141	0.052	0.011	0.405	#13	0.473	Not Required	Pass
301	0.021	0.824	0.001	0.048	0.000	0.835	#13	0.540	Not Required	Pass
302	0.002	0.221	0.515	0.053	0.104	0.737	#13	0.035	Not Required	Pass
303	0.007	0.811	0.045	0.081	0.007	0.852	#13	0.045	Not Required	Pass
304	0.007	0.810	0.106	0.081	0.023	0.873	#13	0.080	Not Required	Pass
305	0.007	0.505	0.110	0.081	0.027	0.529	#13	0.074	Not Required	Pass
306	0.007	0.772	0.045	0.076	0.007	0.814	#13	0.045	Not Required	Pass
307	0.007	0.480	0.110	0.077	0.027	0.505	#13	0.074	Not Required	Pass
308	0.002	0.083	0.096	0.052	0.011	0.118	#13	0.095	Not Required	Pass
309	0.008	0.026	0.085	0.001	0.001	0.109	#13	0.204	Not Required	Pass
310	0.007	0.785	0.106	0.078	0.023	0.855	#13	0.080	Not Required	Pass
311	0.001	0.092	0.098	0.050	0.011	0.121	#15	0.095	Not Required	Pass
312	0.002	0.210	0.483	0.052	0.100	0.694	#13	0.035	Not Required	Pass
313	0.004	0.223	0.254	0.065	0.014	0.421	#13	0.286	Not Required	Pass
314	0.006	0.245	0.252	0.067	0.014	0.435	#13	0.286	Not Required	Pass
315	0.003	0.298	0.141	0.051	0.011	0.419	#13	0.473	Not Required	Pass
316	0.002	0.285	0.141	0.051	0.011	0.407	#13	0.473	Not Required	Pass
401	0.019	0.733	0.006	0.042	0.000	0.744	#13	0.540	Not Required	Pass
402	0.002	0.223	0.505	0.052	0.100	0.728	#13	0.035	Not Required	Pass
403	0.007	0.778	0.050	0.079	0.010	0.825	#13	0.045	Not Required	Pass
404	0.007	0.758	0.104	0.076	0.022	0.812	#13	0.080	Not Required	Pass
405	0.007	0.484	0.109	0.077	0.027	0.507	#13	0.074	Not Required	Pass
406	0.006	0.640	0.030	0.063	0.004	0.668	#13	0.045	Not Required	Pass
407	0.005	0.398	0.081	0.064	0.020	0.415	#13	0.074	Not Required	Pass
408	0.000	0.072	0.089	0.031	0.007	0.149	#13	Not Required	Not Required	Pass
409	0.007	0.036	0.098	0.002	0.002	0.121	#15	0.204	Not Required	Pass
410	0.005	0.640	0.080	0.064	0.017	0.698	#13	0.080	Not Required	Pass
411	0.000	0.072	0.089	0.031	0.007	0.149	#13	Not Required	Not Required	Pass
412	0.002	0.159	0.384	0.041	0.082	0.544	#13	0.035	Not Required	Pass
413	0.004	0.237	0.253	0.067	0.014	0.417	#15	0.190	Not Required	Pass
414	0.005	0.232	0.251	0.066	0.014	0.398	#15	0.286	Not Required	Pass
415	0.002	0.320	0.141	0.053	0.011	0.441	#13	0.473	Not Required	Pass
416	0.002	0.320	0.141	0.051	0.011	0.441	#13	0.473	Not Required	Pass

Definitions

Φ_t	Safety factor for tensile
Φ_c	Safety factor for compression
Φ_b	Safety factor for flexure
Φ_v	Safety factor for shear
E	Modulus of elasticity
F_y	Specified minimum yield stress
F_u	Specified minimum tensile strength

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

REFERENCES	CALCULATIONS	RESULTS
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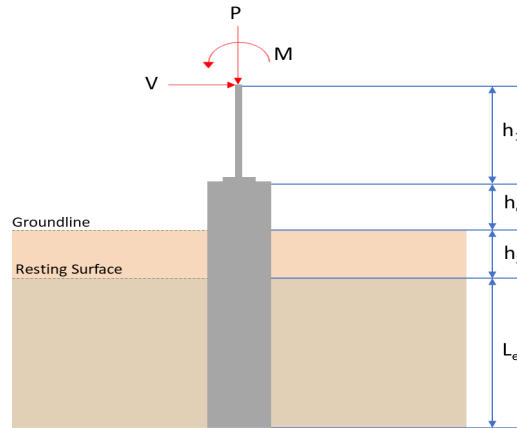
SkyCiv Foundation Design

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

$L = 11$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.922	6.710
V_x (kip)	-6.467	-10.778
V_z (kip)	0.083	0.124
M_x (kipft)	0.347	0.515
M_z (kipft)	100.276	168.263

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,

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

$$H_o = \frac{(-6.467 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0298 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(100.28 \text{ kipft}) + ((-6.467 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.968 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.9727 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.083 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.013217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.347 \text{ kipft}) + ((0.083 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.055255 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0889 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(9.9727 \text{ ft}), (2.0889 \text{ ft})]$$

$$L_{e,req} = 9.973 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (11 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 11 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(9.973 \text{ ft})}{(11 \text{ ft})}$$

$$\text{Ratio} = 0.90664$$

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

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

$$q = \frac{(4.922 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.30763 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.30763 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.20508$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(11 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.75$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.0298 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 15.968 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.968 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (15.968 \text{ kipft/ft})) + (4 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.6277 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (15.968 \text{ kipft/ft})) + (3 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]^2}{(11 \text{ ft})^2 \times [(3 \times (15.968 \text{ kipft/ft})) + (2 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]}$$

$$p = 0.2193 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (15.968 \text{ kipft/ft})) + ((-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]}{(11 \text{ ft})^2}$$

$$s = 1.0219 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.6277 \text{ ft})}{2}$$

$$p_a = 0.38138 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2193 \text{ kip/ft}^2)}{(0.38138 \text{ kip/ft}^2)}$$

$$Ratio = 0.57501$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11 \text{ ft})$$

$$p_s = 1.1 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0219 \text{ kip/ft}^2)}{(1.1 \text{ kip/ft}^2)}$$

$$Ratio = 0.92896$$

Status: **PASS**
Ratio: **0.580**

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

Considering z-direction:

$H_o = 0.013217 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.055255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.055255 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (0.013217 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (0.055255 \text{ kipft/ft})) + (4 \times (0.013217 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.9172 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.055255 \text{ kipft/ft})) + (3 \times (0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]^2}{(11 \text{ ft})^2 \times [(3 \times (0.055255 \text{ kipft/ft})) + (2 \times (0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]}$$

$$p = 0.0058635 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.055255 \text{ kipft/ft})) + ((0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]}{(11 \text{ ft})^2}$$

$$s = 0.012689 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.9172 \text{ ft})}{2}$$

$$p_a = 0.39586 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0058635 \text{ kip/ft}^2)}{(0.39586 \text{ kip/ft}^2)}$$

$$Ratio = 0.014812$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11 \text{ ft})$$

$$p_s = 1.1 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

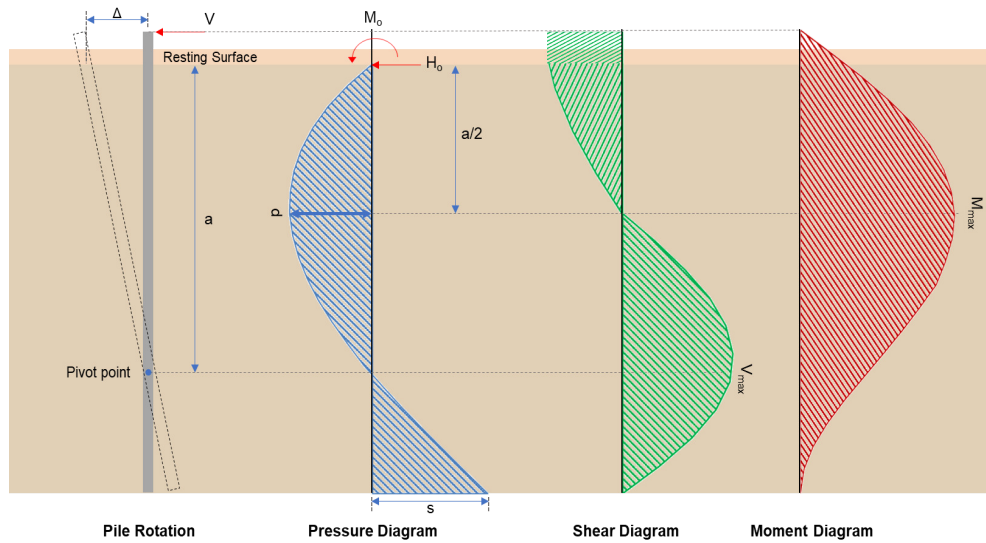
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.012689 \text{ kip/ft}^2)}{(1.1 \text{ kip/ft}^2)}$$

$$Ratio = 0.011535$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-10.778 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.7162 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(168.26 \text{ kipft}) + ((-10.778 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 26.793 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(26.793 \text{ kipft/ft})}{(-1.7162 \text{ kip/ft})}$$

$$E = 15.612 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (26.793 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-1.7162 \text{ kip/ft}) \times (11 \text{ ft})^2)}{6 \times (26.793 \text{ kipft/ft}) + 4 \times (-1.7162 \text{ kip/ft}) \times 11 \text{ ft}}$$

$$a = \frac{6 \times (26.793 \text{ kipft/ft}) + (4 \times (-1.7162 \text{ kip/ft}) \times (11 \text{ ft}))}{\dots}$$

$$a = 7.6263 \text{ ft}$$

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

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

$$V_{max} = ((-1.7162 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.6263 \text{ ft})}{(11 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.6263 \text{ ft})}{(11 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.767 \text{ kip}$$

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

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

$$M_{max} = ((-1.7162 \text{ kip/ft}) \times (48 \text{ in}) \times (11 \text{ ft})) \times \left[\left(\frac{(15.612 \text{ ft})}{(11 \text{ ft})} + \frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 112.88 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.124 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.019745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.515 \text{ kipft}) + ((0.124 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.082006 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.082006 \text{ kipft/ft})}{(0.019745 \text{ kip/ft})}$$

$$E = 4.1532 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.082006 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (0.019745 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (0.082006 \text{ kipft/ft})) + (4 \times (0.019745 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.9186 \text{ ft}$$

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

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

$$V_{max} = ((0.019745 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1532 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.9186 \text{ ft})}{(11 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.1532 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.9186 \text{ ft})}{(11 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10562 \text{ kip}$$

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

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

$$M_{max} = ((0.019745 \text{ kip/ft}) \times (48 \text{ in}) \times (11 \text{ ft})) \times \left[\left(\frac{(4.1532 \text{ ft})}{(11 \text{ ft})} + \frac{(7.9186 \text{ ft})}{2 \times (11 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.1532 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.9186 \text{ ft})}{2 \times (11 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1532 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.9186 \text{ ft})}{2 \times (11 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.50369 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.71 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.373 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.373 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: 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;">$Ratio = \frac{(6.71 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0025082$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.71 \text{ kip} \rightarrow 6710 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6710 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.38 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.38 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.38 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.38 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.68 \text{ kip}$$

Considering x-direction:

$V_{max} = 21.767 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.767 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.19667$$

Status: **PASS**
Ratio: **0.200**

Considering z-direction:

$V_{max} = 0.10562 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.10562 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.00095429$$

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:

$M_{max} = 112.88 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(112.88 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.45224$$

Status: **PASS**
Ratio: **0.450**

Considering z-direction:

$M_{max} = 0.50369 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.50369 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.002018$$

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

REFERENCES	CALCULATIONS	RESULTS
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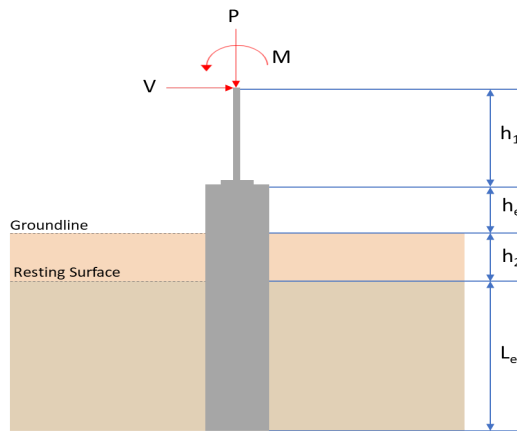
SkyCiv Foundation Design

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
 $L = 11$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	4.922	6.710
V_x (kip)	-6.467	-10.778
V_z (kip)	-0.083	-0.124
M_x (kipft)	-0.347	-0.516
M_z (kipft)	100.276	168.264

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,

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

$$H_o = \frac{(-6.467 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.0298 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(100.28 \text{ kipft}) + ((-6.467 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 15.968 \text{ kipft/ft}$$

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:

$L_{e,x} = 9.9727 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.083 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.013217 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.347 \text{ kipft}) + ((-0.083 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.055255 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6685 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(9.9727 \text{ ft}), (1.6685 \text{ ft})]$$

$$L_{e,req} = 9.973 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (11 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 11 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(9.973 \text{ ft})}{(11 \text{ ft})}$$

$$\text{Ratio} = 0.90664$$

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

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

$$q = \frac{(4.922 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.30763 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.30763 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.20508$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(11 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.75$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.0298 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 15.968 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (15.968 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (15.968 \text{ kipft/ft})) + (4 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.6277 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (15.968 \text{ kipft/ft})) + (3 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]^2}{(11 \text{ ft})^2 \times [(3 \times (15.968 \text{ kipft/ft})) + (2 \times (-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]}$$

$$p = 0.2193 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (15.968 \text{ kipft/ft})) + ((-1.0298 \text{ kip/ft}) \times (11 \text{ ft}))]}{(11 \text{ ft})^2}$$

$$s = 1.0219 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.6277 \text{ ft})}{2}$$

$$p_a = 0.38138 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.2193 \text{ kip/ft}^2)}{(0.38138 \text{ kip/ft}^2)}$$

$$Ratio = 0.57501$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11 \text{ ft})$$

$$p_s = 1.1 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0219 \text{ kip/ft}^2)}{(1.1 \text{ kip/ft}^2)}$$

$$Ratio = 0.92896$$

Status: **PASS**
Ratio: **0.580**

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

Considering z-direction:

$H_o = -0.013217 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.055255 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.055255 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-0.013217 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (0.055255 \text{ kipft/ft})) + (4 \times (-0.013217 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.9172 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.055255 \text{ kipft/ft})) + (3 \times (-0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]^2}{(11 \text{ ft})^2 [(3 \times (0.055255 \text{ kipft/ft})) + (2 \times (-0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]}$$

$$p = -0.0022949 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.055255 \text{ kipft/ft})) + ((-0.013217 \text{ kip/ft}) \times (11 \text{ ft}))]}{(11 \text{ ft})^2}$$

$$s = -0.0017292 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.9172 \text{ ft})}{2}$$

$$p_a = 0.39586 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0022949 \text{ kip/ft}^2)}{(0.39586 \text{ kip/ft}^2)}$$

$$Ratio = -0.0057972$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11 \text{ ft})$$

$$p_s = 1.1 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

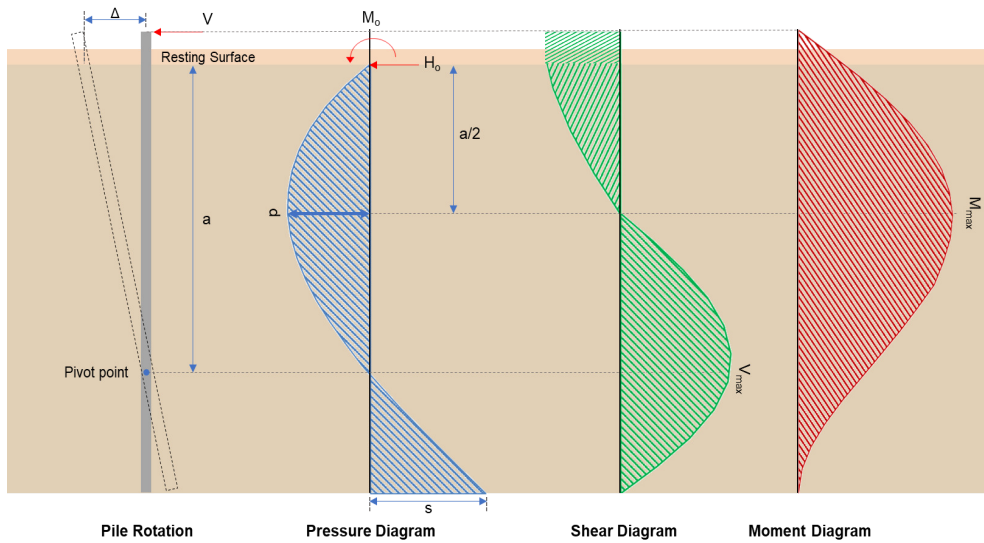
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0017292 \text{ kip/ft}^2)}{(1.1 \text{ kip/ft}^2)}$$

$$Ratio = -0.001572$$

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,

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

$$H_o = \frac{(-10.778 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.7162 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(168.26 \text{ kipft}) + ((-10.778 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 26.794 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(26.794 \text{ kipft/ft})}{(-1.7162 \text{ kip/ft})}$$

$$E = 15.612 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (26.794 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-1.7162 \text{ kip/ft}) \times (11 \text{ ft})^2)}{6 \times (26.794 \text{ kipft/ft}) + 4 \times (-1.7162 \text{ kip/ft}) \times (11 \text{ ft})}$$

$$a = \frac{(6 \times (26.794 \text{ kipft/ft})) + (4 \times (-1.7162 \text{ kip/ft}) \times (11 \text{ ft}))}{\dots}$$

$$a = 7.6263 \text{ ft}$$

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

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

$$V_{max} = ((-1.7162 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.6263 \text{ ft})}{(11 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.6263 \text{ ft})}{(11 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 21.767 \text{ kip}$$

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

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

$$M_{max} = ((-1.7162 \text{ kip/ft}) \times (48 \text{ in}) \times (11 \text{ ft})) \times \left[\left(\frac{(15.612 \text{ ft})}{(11 \text{ ft})} + \frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.612 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.6263 \text{ ft})}{2 \times (11 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 112.88 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.124 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.019745 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.516 \text{ kipft}) + ((-0.124 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.082166 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.082166 \text{ kipft/ft})}{(-0.019745 \text{ kip/ft})}$$

$$E = 4.1613 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.082166 \text{ kipft/ft}) \times (11 \text{ ft})) + (3 \times (-0.019745 \text{ kip/ft}) \times (11 \text{ ft})^2)}{(6 \times (0.082166 \text{ kipft/ft})) + (4 \times (-0.019745 \text{ kip/ft}) \times (11 \text{ ft}))}$$

$$a = 7.9181 \text{ ft}$$

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

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

$$V_{max} = ((-0.019745 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.1613 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.9181 \text{ ft})}{(11 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.1613 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.9181 \text{ ft})}{(11 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.10572 \text{ kip}$$

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

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

$$M_{max} = ((-0.019745 \text{ kip/ft}) \times (48 \text{ in}) \times (11 \text{ ft})) \times \left[\left(\frac{(4.1613 \text{ ft})}{(11 \text{ ft})} + \frac{(7.9181 \text{ ft})}{2 \times (11 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.1613 \text{ ft})}{(11 \text{ ft})} + 3 \right) \times \left(\frac{(7.9181 \text{ ft})}{2 \times (11 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.1613 \text{ ft})}{(11 \text{ ft})} + 2 \right) \times \left(\frac{(7.9181 \text{ ft})}{2 \times (11 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.50425 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(0.71 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.373 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.373 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: 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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</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;">$Ratio = \frac{(6.71 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0025082$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.71 \text{ kip} \rightarrow 6710 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6710 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.38 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.38 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.38 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.38 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.68 \text{ kip}$$

Considering x-direction:

$V_{max} = 21.767 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(21.767 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.19667$$

Status: **PASS**
Ratio: **0.200**

Considering z-direction:

$V_{max} = 0.10572 \text{ kip}$ - Maximum shear force in the z-direction,
 $Ratio$ - Capacity

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

$$Ratio = \frac{(0.10572 \text{ kip})}{(110.68 \text{ kip})}$$

$$Ratio = 0.0009552$$

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:

$M_{max} = 112.88 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(112.88 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.45225$$

Status: **PASS**
Ratio: **0.450**

Considering z-direction:

$M_{max} = 0.50425 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.50425 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0020202$$

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

REFERENCES	CALCULATIONS	RESULTS
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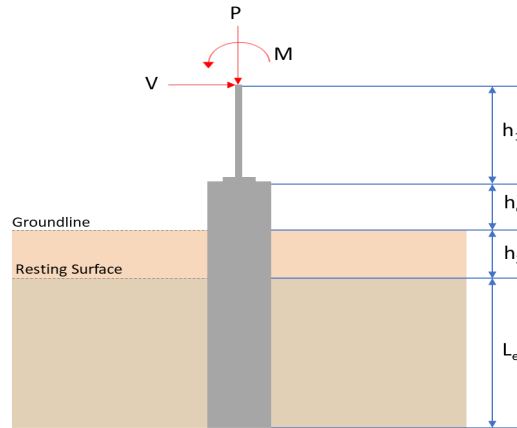
SkyCiv Foundation Design

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

$L = 11.25$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.438	7.440
V_x (kip)	-7.286	-12.140
V_z (kip)	0.012	0.020
M_x (kipft)	-0.045	0.075
M_z (kipft)	112.821	189.346

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,

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

$$H_o = \frac{(-7.286 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1602 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(112.82 \text{ kipft}) + ((-7.286 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 17.965 \text{ kipft/ft}$$

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:

$L_{e,x} = 10.271 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.045 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0071656 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0113 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(10.271 \text{ ft}), (1.0113 \text{ ft})]$$

$$L_{e,req} = 10.271 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (11.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 11.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(10.271 \text{ ft})}{(11.25 \text{ ft})}$$

$$\text{Ratio} = 0.91298$$

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

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

$$q = \frac{(5.438 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33988 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33988 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.22658$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(11.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.1602 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 17.965 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (17.965 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (17.965 \text{ kipft/ft})) + (4 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 7.8059 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (17.965 \text{ kipft/ft})) + (3 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]^2}{(11.25 \text{ ft})^2 \times [(3 \times (17.965 \text{ kipft/ft})) + (2 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]}$$

$$p = 0.22806 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (17.965 \text{ kipft/ft})) + ((-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]}{(11.25 \text{ ft})^2}$$

$$s = 1.0846 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.8059 \text{ ft})}{2}$$

$$p_a = 0.3903 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22806 \text{ kip/ft}^2)}{(0.3903 \text{ kip/ft}^2)}$$

$$Ratio = 0.58433$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11.25 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0846 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.96408$$

Status: **PASS**
Ratio: **0.580**

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

Considering z-direction:

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

$M_o = 0.0071656 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0071656 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (0.0071656 \text{ kipft/ft})) + (4 \times (0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 8.125 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0071656 \text{ kipft/ft})) + (3 \times (0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]^2}{(11.25 \text{ ft})^2 \times [(3 \times (0.0071656 \text{ kipft/ft})) + (2 \times (0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]}$$

$$p = 0.00079736 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0071656 \text{ kipft/ft})) + ((0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]}{(11.25 \text{ ft})^2}$$

$$s = 0.0016985 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(8.125 \text{ ft})}{2}$$

$$p_a = 0.40625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.00079736 \text{ kip/ft}^2)}{(0.40625 \text{ kip/ft}^2)}$$

$$Ratio = 0.0019627$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11.25 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

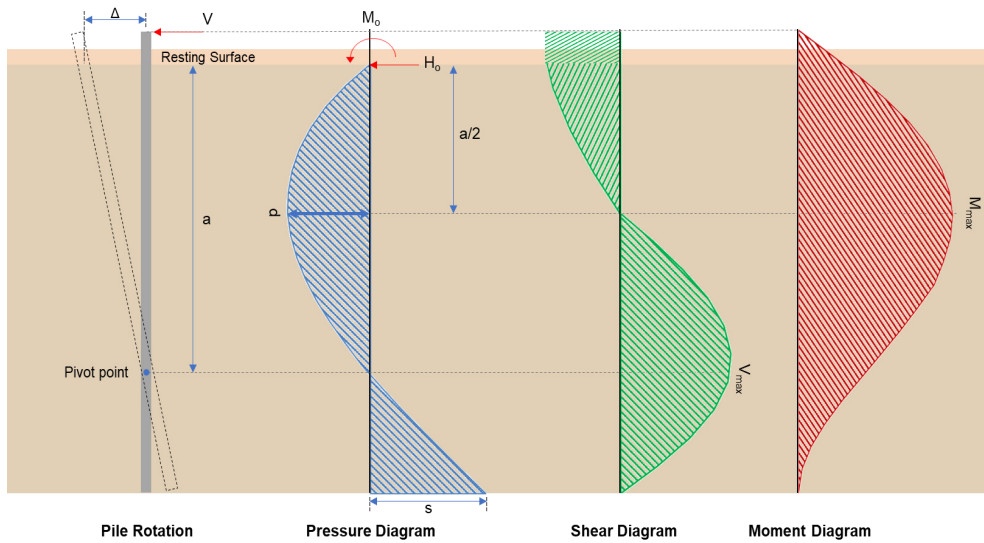
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0016985 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.0015098$$

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,

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

$$H_o = \frac{(-12.14 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.9331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(189.35 \text{ kipft}) + ((-12.14 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 30.151 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(30.151 \text{ kipft/ft})}{(-1.9331 \text{ kip/ft})}$$

$$E = 15.597 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (30.151 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (30.151 \text{ kipft/ft})) + (4 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = \frac{(6 \times (30.151 \text{ kip/ft})) + (4 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft}))}{}$$

$$a = 7.8044 \text{ ft}$$

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

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

$$V_{max} = ((-1.9331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.8044 \text{ ft})}{(11.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.8044 \text{ ft})}{(11.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 24.068 \text{ kip}$$

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

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

$$M_{max} = ((-1.9331 \text{ kip/ft}) \times (48 \text{ in}) \times (11.25 \text{ ft})) \times \left[\left(\frac{(15.597 \text{ ft})}{(11.25 \text{ ft})} + \frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 127.51 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.02 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0031847 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075 \text{ kipft}) + ((0.02 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011943 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.011943 \text{ kipft/ft})}{(0.0031847 \text{ kip/ft})}$$

$$E = 3.75 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011943 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (0.0031847 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (0.011943 \text{ kipft/ft})) + (4 \times (0.0031847 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 8.125 \text{ ft}$$

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

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

$$V_{max} = ((0.0031847 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(8.125 \text{ ft})}{(11.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(8.125 \text{ ft})}{(11.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.016055 \text{ kip}$$

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

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

$$M_{max} = ((0.0031847 \text{ kip/ft}) \times (48 \text{ in}) \times (11.25 \text{ ft})) \times \left[\left(\frac{(3.75 \text{ ft})}{(11.25 \text{ ft})} + \frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.07759 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.44 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.349 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.349 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(7.44 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0027811$	<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> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.44 \text{ kip} \rightarrow 7440 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7440 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.48 \text{ kip}$$

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

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (119.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.48 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.48 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.74 \text{ kip}$$

Considering x-direction:

$V_{max} = 24.068 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(24.068 \text{ kip})}{(110.74 \text{ kip})}$$

$$Ratio = 0.21734$$

Status: **PASS**
Ratio: **0.220**

Considering z-direction:

$V_{max} = 0.016055 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.016055 \text{ kip})}{(110.74 \text{ kip})}$$

$$Ratio = 0.00014497$$

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:

$M_{max} = 127.51 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(127.51 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.51085$$

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

Considering z-direction:

$M_{max} = 0.07759 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.07759 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00031086$$

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

REFERENCES	CALCULATIONS	RESULTS
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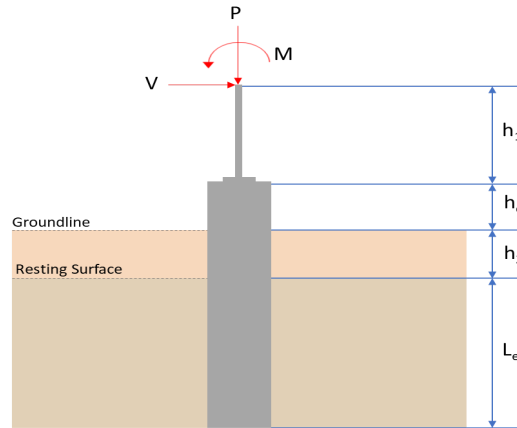
SkyCiv Foundation Design

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

$L = 11.25$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.455	7.470
V_x (kip)	-7.377	-12.295
V_z (kip)	0.000	0.000
M_x (kipft)	0.000	0.000
M_z (kipft)	114.438	192.117

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,

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

$$H_o = \frac{(-7.377 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1747 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(114.44 \text{ kipft}) + ((-7.377 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 18.223 \text{ kipft/ft}$$

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:

$L_{e,x} = 10.312 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(10.312 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 10.312 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (11.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 11.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(10.312 \text{ ft})}{(11.25 \text{ ft})}$$

$$\text{Ratio} = 0.91662$$

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

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

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

$$q = \frac{(5.455 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.34094 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.34094 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.22729$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(11.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.1747 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 18.223 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (18.223 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.1747 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (18.223 \text{ kipft/ft})) + (4 \times (-1.1747 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 7.8055 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (18.223 \text{ kipft/ft})) + (3 \times (-1.1747 \text{ kip/ft}) \times (11.25 \text{ ft}))]^2}{(11.25 \text{ ft})^2 \times [(3 \times (18.223 \text{ kipft/ft})) + (2 \times (-1.1747 \text{ kip/ft}) \times (11.25 \text{ ft}))]}$$

$$p = 0.23194 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (18.223 \text{ kipft/ft})) + ((-1.1747 \text{ kip/ft}) \times (11.25 \text{ ft}))]}{(11.25 \text{ ft})^2}$$

$$s = 1.1013 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.8055 \text{ ft})}{2}$$

$$p_a = 0.39028 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23194 \text{ kip/ft}^2)}{(0.39028 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.5943$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11.25 \text{ ft})$$

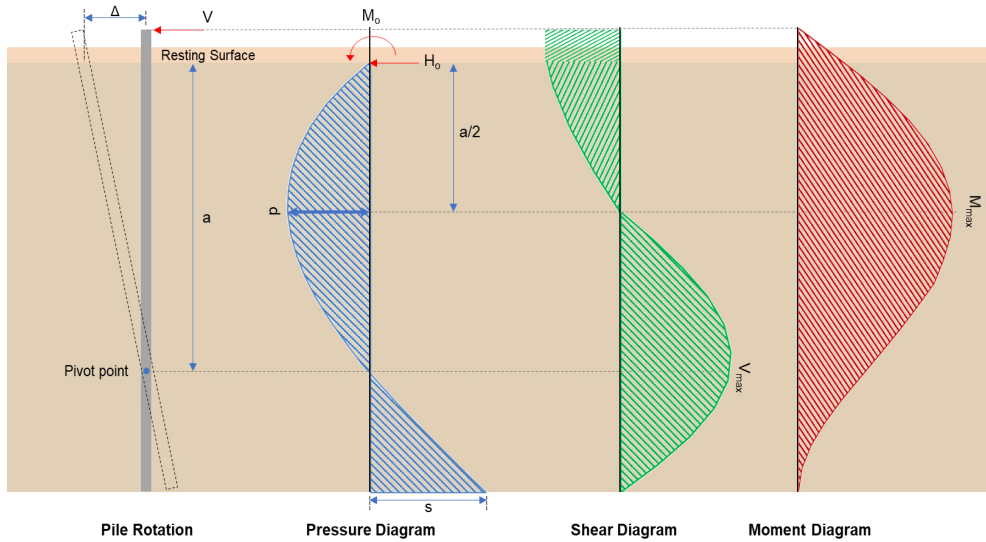
$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.1013 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-12.295 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.9578 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(192.12 \text{ kipft}) + ((-12.295 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 30.592 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(30.592 \text{ kipft/ft})}{(-1.9578 \text{ kip/ft})}$$

$$E = 15.626 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (30.592 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.9578 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (30.592 \text{ kipft/ft})) + (4 \times (-1.9578 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 7.804 \text{ ft}$$

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

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

$$V_{max} = ((-1.9578 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.626 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.804 \text{ ft})}{(11.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (15.626 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.804 \text{ ft})}{(11.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 24.411 \text{ kip}$$

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

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

$$M_{max} = ((-1.9578 \text{ kip/ft}) \times (48 \text{ in}) \times (11.25 \text{ ft})) \times \left[\left(\frac{(15.626 \text{ ft})}{(11.25 \text{ ft})} + \frac{(7.804 \text{ ft})}{2 \times (11.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.626 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.804 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (15.626 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.804 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 129.34 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,

$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,

$\phi = 0.65$ - Reduction factor for axial strength,

$\alpha = 0.8$ - Alpha factor for axial strength,

$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.47 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.348 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.348 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

$$n_{rebar} = \frac{A_{min}}{A_{rebar}}$$

$$n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

$$A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$$

$$A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 4.2951 \text{ in}^2$$

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

$$s_{rebar} = \text{Max} [1.5, (1.5 d_{bar})]$$

Status: **PASS**
Ratio: **0.970**

$$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$$

$$s_{rebar} = 1.5 \text{ in}$$

Ties:

25.7.2.2 Since longitudinal reinforcement is \leq No. 10 \emptyset : Use #3(0.375 in)

25.7.2.1 s_{ties} - Maximum spacing of ties,

$$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$$

$$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$$

$$s_{ties} = 10 \text{ in}$$

Summary:

Main reinforcement: **14 - #5 (0.625 in)**

Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

$$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$$

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$Ratio = \frac{(7.47 \text{ kip})}{(2675.2 \text{ kip})}$$

$$Ratio = 0.0027923$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2 $b_w = 48 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,

22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.47 \text{ kip} \rightarrow 7470 \text{ lbf}$,

22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(7470 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.48 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.48 \text{ kip}$$

22.5.1.2 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

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

$$\phi V_n = (0.65) \times ((119.48 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.74 \text{ kip}$$

Considering x-direction:

$V_{max} = 24.411 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(24.411 \text{ kip})}{(110.74 \text{ kip})}$$

$$\text{Ratio} = 0.22043$$

Status: **PASS**
Ratio: **0.220**

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 $\phi M_{n,2}$

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

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

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

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: $M_{max} = 129.34 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(129.34 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.51817$$

Status: **PASS**
Ratio: **0.520**

REFERENCES	CALCULATIONS	RESULTS
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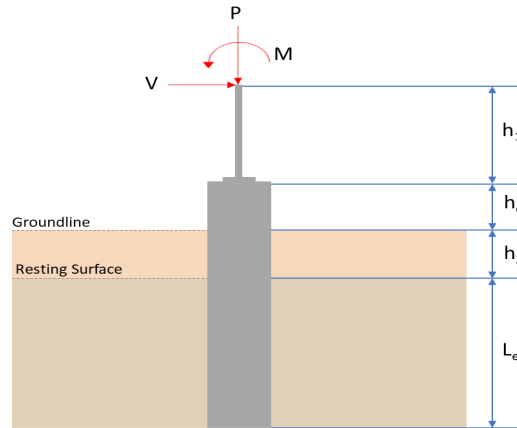
SkyCiv Foundation Design

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

$L = 11.25$ 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	Clay, sandy clay, silty clay, clayey silt, silt and sandy silt	1500.000	100.000

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	5.438	7.440
V_x (kip)	-7.286	-12.140
V_z (kip)	-0.012	-0.020
M_x (kipft)	0.045	-0.075
M_z (kipft)	112.821	189.346

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,

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

$$H_o = \frac{(-7.286 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.1602 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(112.82 \text{ kipft}) + ((-7.286 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 17.965 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 10.271 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

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

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

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.045 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0071656 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.89096 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(10.271 \text{ ft}), (0.89096 \text{ ft})]$$

$$L_{e,req} = 10.271 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (11.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 11.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(10.271 \text{ ft})}{(11.25 \text{ ft})}$$

$$\text{Ratio} = 0.91298$$

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

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

$$q = \frac{(5.438 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.33988 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.33988 \text{ kip/ft}^2)}{(1500 \text{ psf})}$$

$$\text{Ratio} = 0.22658$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(11.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 2.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -1.1602 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 17.965 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (17.965 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (17.965 \text{ kipft/ft})) + (4 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 7.8059 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (17.965 \text{ kipft/ft})) + (3 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]^2}{(11.25 \text{ ft})^2 \times [(3 \times (17.965 \text{ kipft/ft})) + (2 \times (-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]}$$

$$p = 0.22806 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (17.965 \text{ kipft/ft})) + ((-1.1602 \text{ kip/ft}) \times (11.25 \text{ ft}))]}{(11.25 \text{ ft})^2}$$

$$s = 1.0846 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(7.8059 \text{ ft})}{2}$$

$$p_a = 0.3903 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.22806 \text{ kip/ft}^2)}{(0.3903 \text{ kip/ft}^2)}$$

$$Ratio = 0.58433$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11.25 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(1.0846 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = 0.96408$$

Status: **PASS**
Ratio: **0.580**

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

Considering z-direction:

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

$M_o = 0.0071656 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0071656 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (0.0071656 \text{ kipft/ft})) + (4 \times (-0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 8.125 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.0071656 \text{ kipft/ft})) + (3 \times (-0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]^2}{(11.25 \text{ ft})^2 \times [(3 \times (0.0071656 \text{ kipft/ft})) + (2 \times (-0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]}$$

$$p = -0.00035386 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.0071656 \text{ kipft/ft})) + ((-0.0019108 \text{ kip/ft}) \times (11.25 \text{ ft}))]}{(11.25 \text{ ft})^2}$$

$$s = -0.0003397 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (100 \text{ psf/ft}) \times \frac{(8.125 \text{ ft})}{2}$$

$$p_a = 0.40625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.00035386 \text{ kip/ft}^2)}{(0.40625 \text{ kip/ft}^2)}$$

$$Ratio = -0.00087103$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (100 \text{ psf/ft}) \times (11.25 \text{ ft})$$

$$p_s = 1.125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

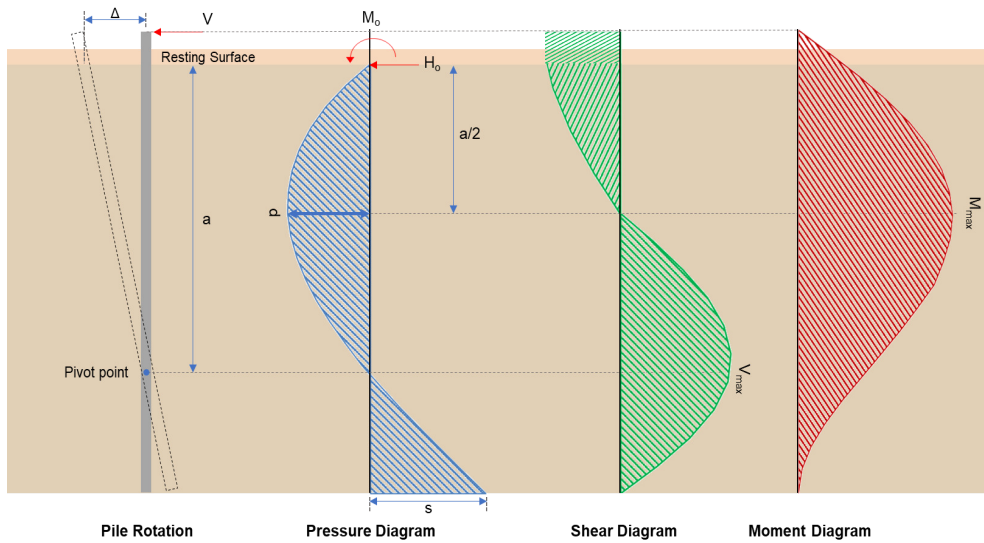
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(-0.0003397 \text{ kip/ft}^2)}{(1.125 \text{ kip/ft}^2)}$$

$$Ratio = -0.00030196$$

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,

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

$$H_o = \frac{(-12.14 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -1.9331 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(189.35 \text{ kipft}) + ((-12.14 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 30.151 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(30.151 \text{ kipft/ft})}{(-1.9331 \text{ kip/ft})}$$

$$E = 15.597 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (30.151 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{6 \times (30.151 \text{ kipft/ft}) + 4 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft})}$$

$$a = \frac{(6 \times (30.151 \text{ kipft/ft})) + (4 \times (-1.9331 \text{ kip/ft}) \times (11.25 \text{ ft}))}{}$$

$$a = 7.8044 \text{ ft}$$

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

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

$$V_{max} = ((-1.9331 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.8044 \text{ ft})}{(11.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.8044 \text{ ft})}{(11.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 24.068 \text{ kip}$$

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

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

$$M_{max} = ((-1.9331 \text{ kip/ft}) \times (48 \text{ in}) \times (11.25 \text{ ft})) \times \left[\left(\frac{(15.597 \text{ ft})}{(11.25 \text{ ft})} + \frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.597 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(7.8044 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 127.51 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.02 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0031847 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.075 \text{ kipft}) + ((-0.02 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011943 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.011943 \text{ kipft/ft})}{(-0.0031847 \text{ kip/ft})}$$

$$E = 3.75 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011943 \text{ kipft/ft}) \times (11.25 \text{ ft})) + (3 \times (-0.0031847 \text{ kip/ft}) \times (11.25 \text{ ft})^2)}{(6 \times (0.011943 \text{ kipft/ft})) + (4 \times (-0.0031847 \text{ kip/ft}) \times (11.25 \text{ ft}))}$$

$$a = 8.125 \text{ ft}$$

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

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

$$V_{max} = ((-0.0031847 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(8.125 \text{ ft})}{(11.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(8.125 \text{ ft})}{(11.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.016055 \text{ kip}$$

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

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

$$M_{max} = ((-0.0031847 \text{ kip/ft}) \times (48 \text{ in}) \times (11.25 \text{ ft})) \times \left[\left(\frac{(3.75 \text{ ft})}{(11.25 \text{ ft})} + \frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 3 \right) \times \left(\frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.75 \text{ ft})}{(11.25 \text{ ft})} + 2 \right) \times \left(\frac{(8.125 \text{ ft})}{2 \times (11.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.07759 \text{ kipft}$$

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, $A_{st,required}$

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.44 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.349 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.349 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: 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;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</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;">$Ratio = \frac{(7.44 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0027811$</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 $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 7.44 \text{ kip} \rightarrow 7440 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(7440 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.48 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.48 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.48 \text{ kip}$$

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

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

$$\phi V_n = (0.65) \times ((119.48 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.74 \text{ kip}$$

Considering x-direction:

$V_{max} = 24.068 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(24.068 \text{ kip})}{(110.74 \text{ kip})}$$

$$Ratio = 0.21734$$

Status: **PASS**
Ratio: **0.220**

Considering z-direction:

$V_{max} = 0.016055 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.016055 \text{ kip})}{(110.74 \text{ kip})}$$

$$Ratio = 0.00014497$$

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:

$M_{max} = 127.51 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(127.51 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.51085$$

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

Considering z-direction:

$M_{max} = 0.07759 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.07759 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00031086$$

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