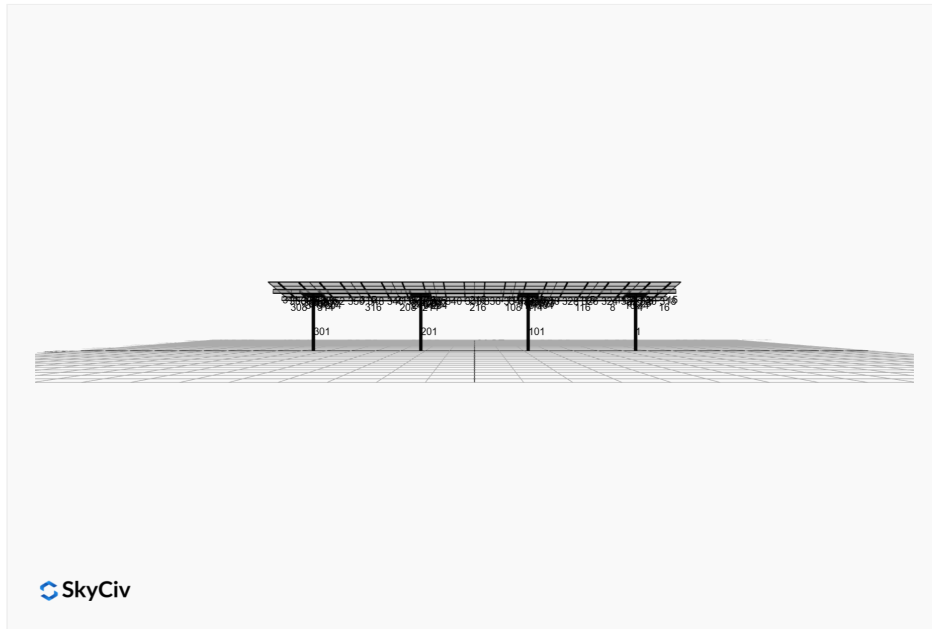


Project Details



Project Name: MTSOLAR_CIIA5936GCG5 **Date:** Wed Oct 30 2024
Location: Fruitland, MD 21826, USA **Number of Modules:** 50
Unique ID: 4P-22.5-6TOP-XD-45-L-5Hx10W-426A **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.96 ft
Array Dimensions E/W	81.67 ft
Winter Tilt Angle	10
Front Edge Clearance	10 ft

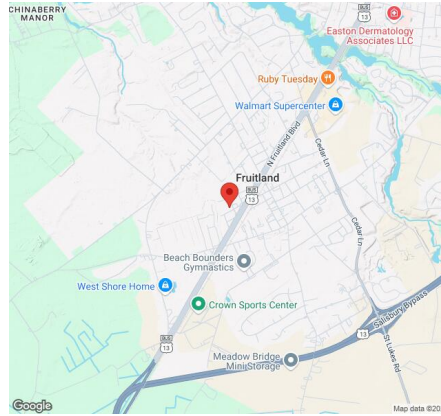
MT Solar Bill of Materials (4P-22.5-6TOP-XD-45-L-5Hx10W-426A)

Part	Short Description	BOM Qty
MTS-PC-6	6IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-45	45IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	12
MTS-CLAMP-ANGLE-4PK	Angle Clamp	10

Rail Bill of Materials

Part	Qty
Rails (225in)	20
Rail Attachment	80
Module Mid Clamp	80
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: Fruitland, MD 21826, USA

Array Specification

Duty Classification:	XD
Module Width:	45.00 in
Module Length:	97.00in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Winter Tilt Angle:	10
Front Edge Clearance:	10
Total Array Height at Tilt:	13.29 ft
Total Frame Length:	82.50 ft
Frame Weight:	5023 lbs
Array Dimensions N/S:	18.96 ft
Array Dimensions E/W:	81.67 ft
Rail Length:	227.50 in
Rail Spacing:	4.08 ft

Support Specifications

Pole Size:	6in Pipe Sch 40
Pole Length above Grade:	11.65 ft
Number of Poles:	4
Pole Spacing:	22.5 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø36 in
Foundation Depth (below grade):	Pile 1: 8.50 ft Pile 2: 8.75 ft Pile 3: 8.75 ft Pile 4: 8.50 ft
Foundation Volume:	9.032 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	Fruitland, MD 21826, USA
Wind Speed:	110 mph

Snow Load:

20 psf

Design Disclaimer

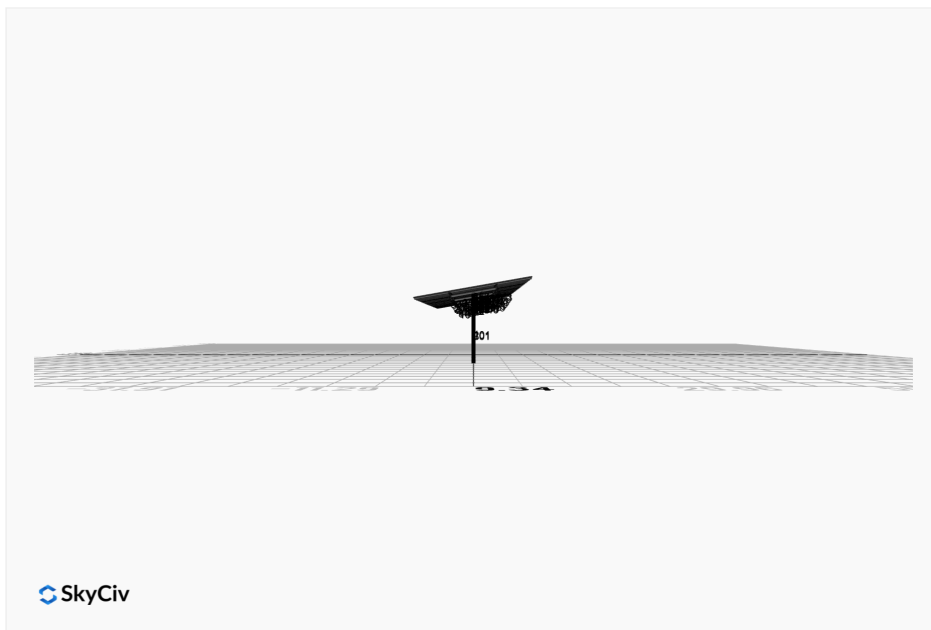
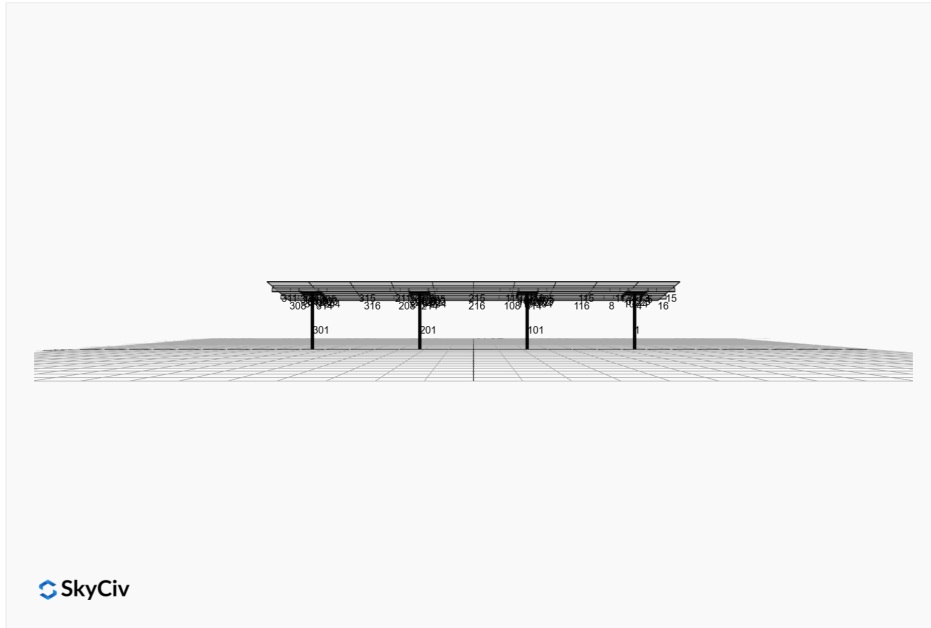
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

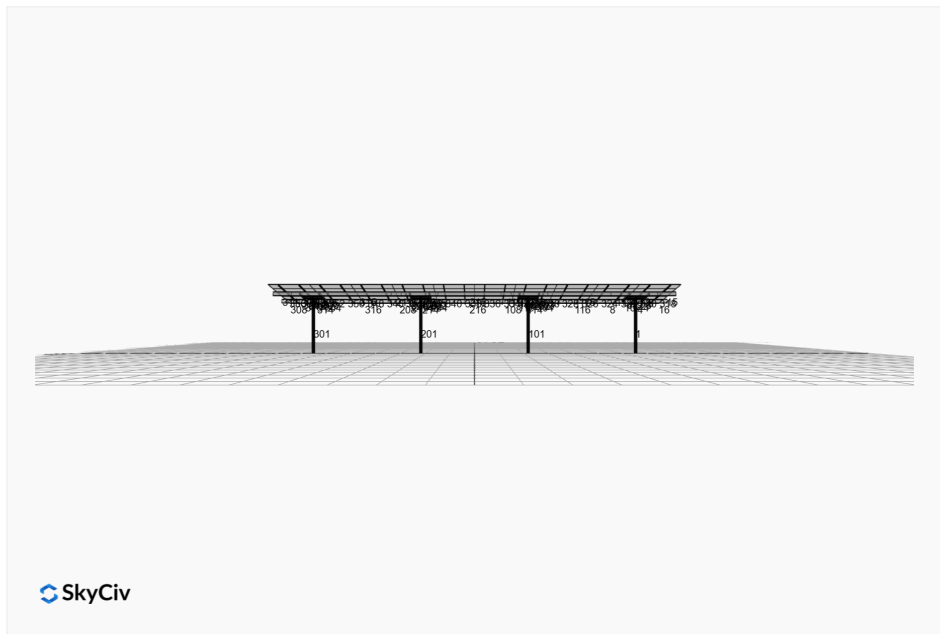
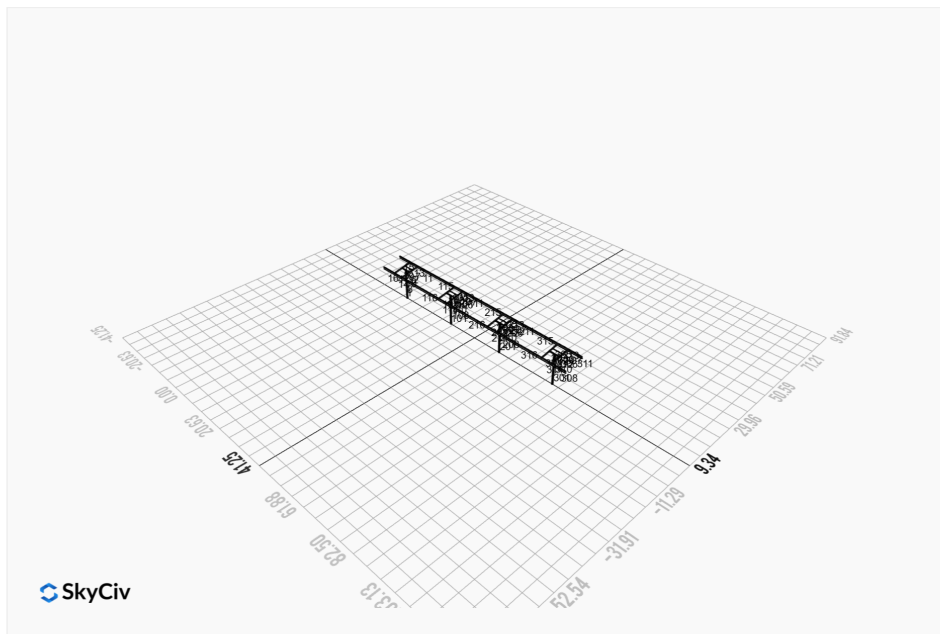
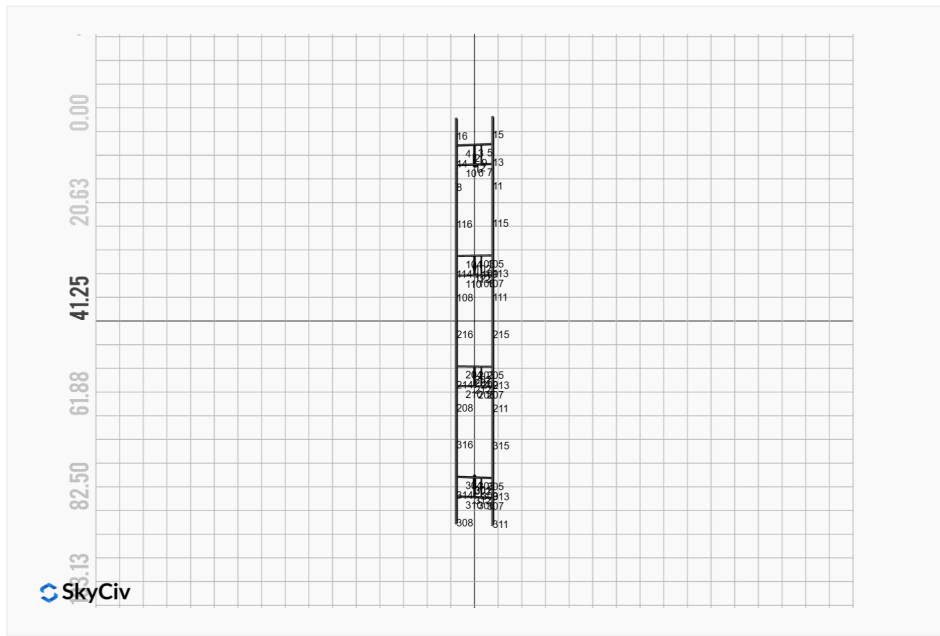
AutoDesigner Input

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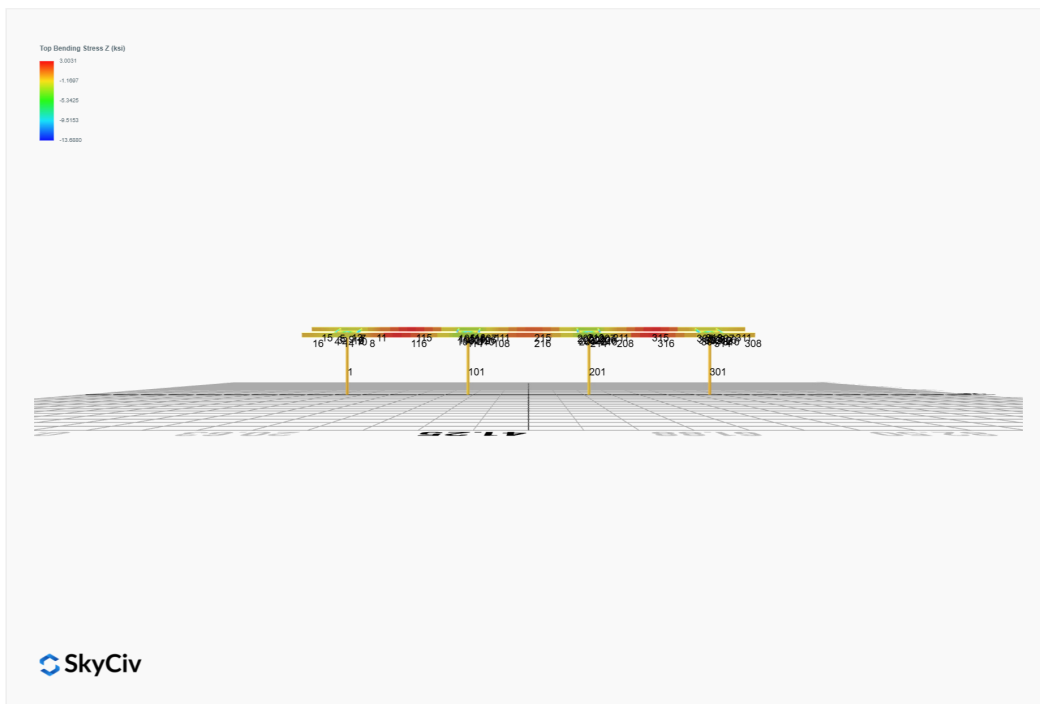
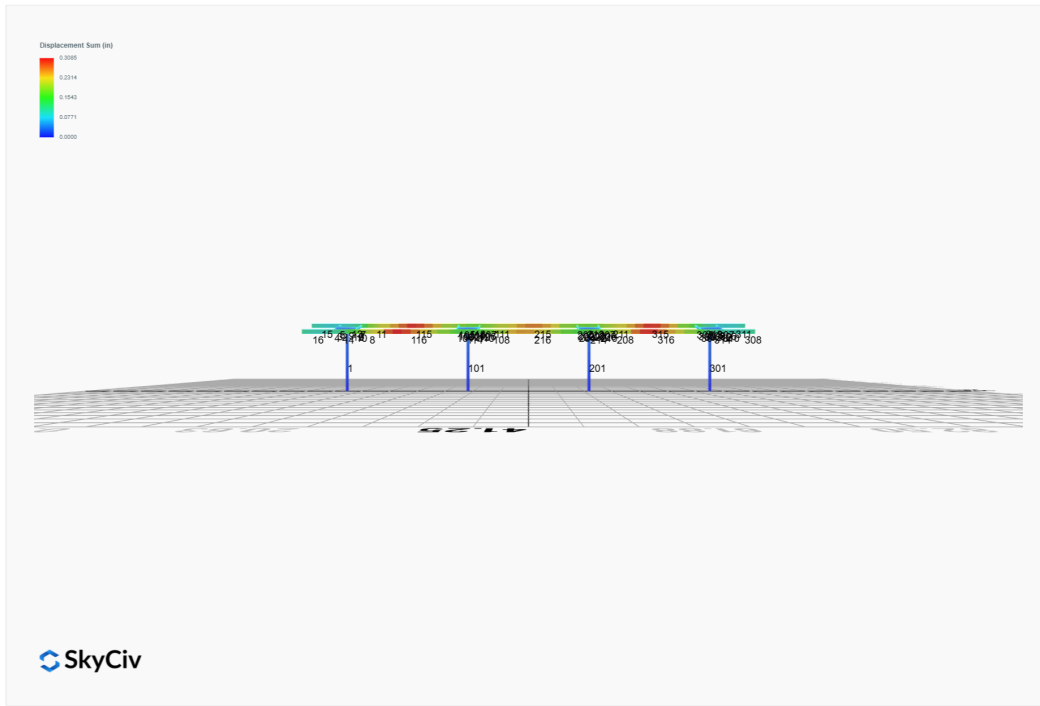
Design Notes:

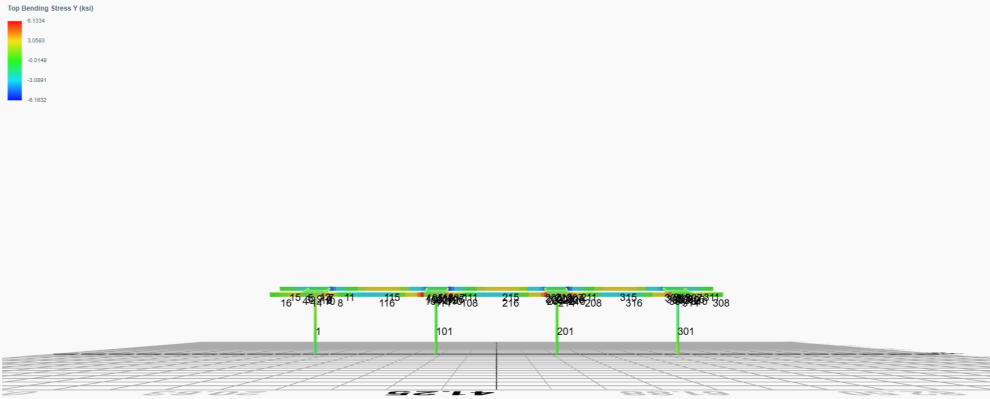
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

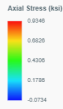




FEM Results (Envelope Worst Case for each member)







SkyCiv

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	0.0091	2.4239	0.0525	0.1959	-0.0099	-0.0601
ULS: 2. D + L	0.0091	2.4239	0.0525	0.1959	-0.0099	-0.0601
ULS: 3. D + (S or Lr or R)	0.0283	6.5232	0.1644	0.6142	-0.0310	-0.2536
ULS: 3. D + (S or Lr or R)	0.0091	2.4239	0.0525	0.1959	-0.0099	-0.0601
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0235	5.4983	0.1364	0.5096	-0.0257	-0.2052
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0091	2.4239	0.0525	0.1959	-0.0099	-0.0601
ULS: 5b. D + 0.7E	0.0091	2.4239	0.0525	0.1959	-0.0099	-0.0601
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0235	5.4983	0.1364	0.5096	-0.0257	-0.2052
ULS: 8. 0.6D + 0.7E	0.0054	1.4543	0.0315	0.1175	-0.0059	-0.0360
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8927	7.3324	0.2065	0.7687	-0.0950	15.5619
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8927	7.3324	0.2065	0.7687	-0.0950	15.5619
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6213	-1.0104	-0.0430	-0.1564	0.0371	-4.1451
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5787	-0.5793	-0.0518	-0.1882	0.0536	-17.6535
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6529	9.1797	0.2519	0.9392	-0.0896	11.5113
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6529	9.1797	0.2519	0.9392	-0.0896	11.5113
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4827	2.9226	0.0648	0.2454	0.0095	-3.2690
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4507	3.2460	0.0582	0.2215	0.0219	-13.4003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6673	6.1053	0.1680	0.6255	-0.0737	11.6564
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6673	6.1053	0.1680	0.6255	-0.0737	11.6564
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4683	-0.1519	-0.0191	-0.0683	0.0254	-3.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4363	0.1715	-0.0258	-0.0922	0.0377	-13.2552
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8964	6.3628	0.1855	0.6904	-0.0911	15.5860
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8964	6.3628	0.1855	0.6904	-0.0911	15.5860
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6177	-1.9800	-0.0640	-0.2348	0.0411	-4.1211
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5751	-1.5488	-0.0729	-0.2666	0.0575	-17.6295

Worst Case Reactions LRFD

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

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

Result	Value (kip, kip-ft)
Axial	13.5589
Shear X	-1.5030
Shear Z	0.3800
Moment X	1.4218
Moment Y (Twist)	0.1685
Moment Z	31.1473

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	9.1797
Shear X	-0.8964
Shear Z	0.2519
Moment X	0.9392
Moment Y (Twist)	0.0950
Moment Z	17.6535

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	-0.0091	2.9501	-0.0087	-0.0328	0.0035	0.1268
ULS: 2. D + L	-0.0091	2.9501	-0.0087	-0.0328	0.0035	0.1268
ULS: 3. D + (S or Lr or R)	-0.0283	8.1666	-0.0274	-0.1027	0.0110	0.3384
ULS: 3. D + (S or Lr or R)	-0.0091	2.9501	-0.0087	-0.0328	0.0035	0.1268
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0235	6.8624	-0.0227	-0.0852	0.0091	0.2855

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0091	2.9501	-0.0087	-0.0328	0.0035	0.1268
ULS: 5b. D + 0.7E	-0.0091	2.9501	-0.0087	-0.0328	0.0035	0.1268
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0235	6.8624	-0.0227	-0.0852	0.0091	0.2855
ULS: 8. 0.6D + 0.7E	-0.0054	1.7700	-0.0052	-0.0197	0.0021	0.0761
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0712	9.1797	-0.0210	-0.0801	-0.0035	18.5385
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0712	9.1797	-0.0210	-0.0801	-0.0035	18.5385
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7560	-1.4268	0.0065	0.0240	-0.0011	-4.9147
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6201	-0.8454	-0.0068	-0.0240	0.0164	-19.8531
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8201	11.5347	-0.0319	-0.1207	0.0038	14.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8201	11.5347	-0.0319	-0.1207	0.0038	14.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5503	3.5798	-0.0113	-0.0426	0.0057	-3.4956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4484	4.0159	-0.0212	-0.0787	0.0188	-14.6994
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8057	7.6223	-0.0179	-0.0683	-0.0018	13.9356
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8057	7.6223	-0.0179	-0.0683	-0.0018	13.9356
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5647	-0.3326	0.0027	0.0098	0.0001	-3.6543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4628	0.1035	-0.0073	-0.0262	0.0132	-14.8581
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0676	7.9997	-0.0175	-0.0670	-0.0050	18.4878
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0676	7.9997	-0.0175	-0.0670	-0.0050	18.4878
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7596	-2.6068	0.0100	0.0371	-0.0025	-4.9654
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6237	-2.0254	-0.0033	-0.0109	0.0150	-19.9038

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	17.0769
Shear X	-1.7841
Shear Z	-0.0493
Moment X	-0.1879
Moment Y (Twist)	0.0326
Moment Z	35.1145

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	11.5347
Shear X	-1.0712
Shear Z	-0.0319
Moment X	-0.1207
Moment Y (Twist)	0.0188
Moment Z	19.9038

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	-0.0091	2.9501	0.0087	0.0328	-0.0035	0.1268
ULS: 2. D + L	-0.0091	2.9501	0.0087	0.0328	-0.0035	0.1268
ULS: 3. D + (S or Lr or R)	-0.0283	8.1666	0.0274	0.1027	-0.0109	0.3384
ULS: 3. D + (S or Lr or R)	-0.0091	2.9501	0.0087	0.0328	-0.0035	0.1268
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0235	6.8624	0.0227	0.0852	-0.0091	0.2855
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0091	2.9501	0.0087	0.0328	-0.0035	0.1268
ULS: 5b. D + 0.7E	-0.0091	2.9501	0.0087	0.0328	-0.0035	0.1268
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0235	6.8624	0.0227	0.0852	-0.0091	0.2855
ULS: 8. 0.6D + 0.7E	-0.0054	1.7700	0.0052	0.0197	-0.0021	0.0761
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.0712	9.1797	0.0210	0.0801	0.0036	18.5385
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.0712	9.1797	0.0210	0.0801	0.0036	18.5385
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.7560	-1.4268	-0.0065	-0.0240	0.0011	-4.9147
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6201	-0.8454	0.0068	0.0240	-0.0164	-19.8531

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8201	11.5347	0.0319	0.1207	-0.0038	14.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8201	11.5347	0.0319	0.1207	-0.0038	14.0943
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5503	3.5798	0.0113	0.0426	-0.0056	-3.4956
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4484	4.0159	0.0212	0.0786	-0.0188	-14.6994
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8057	7.6223	0.0179	0.0683	0.0018	13.9356
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8057	7.6223	0.0179	0.0683	0.0018	13.9356
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5647	-0.3326	-0.0027	-0.0098	-0.0001	-3.6543
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4628	0.1035	0.0073	0.0262	-0.0132	-14.8581
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.0676	7.9997	0.0175	0.0670	0.0050	18.4878
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.0676	7.9997	0.0175	0.0670	0.0050	18.4878
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.7596	-2.6068	-0.0100	-0.0371	0.0025	-4.9654
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6237	-2.0254	0.0033	0.0109	-0.0150	-19.9038

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	17.0769
Shear X	-1.7841
Shear Z	0.0493
Moment X	0.1878
Moment Y (Twist)	0.0325
Moment Z	35.1146

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	11.5347
Shear X	-1.0712
Shear Z	0.0319
Moment X	0.1207
Moment Y (Twist)	0.0188
Moment Z	19.9038

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	0.0091	2.4239	-0.0525	-0.1959	0.0099	-0.0601
ULS: 2. D + L	0.0091	2.4239	-0.0525	-0.1959	0.0099	-0.0601
ULS: 3. D + (S or Lr or R)	0.0283	6.5232	-0.1644	-0.6142	0.0311	-0.2535
ULS: 3. D + (S or Lr or R)	0.0091	2.4239	-0.0525	-0.1959	0.0099	-0.0601
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0235	5.4983	-0.1365	-0.5096	0.0258	-0.2052
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0091	2.4239	-0.0525	-0.1959	0.0099	-0.0601
ULS: 5b. D + 0.7E	0.0091	2.4239	-0.0525	-0.1959	0.0099	-0.0601
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0235	5.4983	-0.1365	-0.5096	0.0258	-0.2052
ULS: 8. 0.6D + 0.7E	0.0054	1.4543	-0.0315	-0.1175	0.0059	-0.0360
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8927	7.3324	-0.2065	-0.7687	0.0950	15.5620
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8927	7.3324	-0.2065	-0.7687	0.0950	15.5620
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6213	-1.0104	0.0430	0.1564	-0.0371	-4.1451
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5787	-0.5793	0.0518	0.1882	-0.0536	-17.6535
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6529	9.1797	-0.2519	-0.9392	0.0896	11.5113
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6529	9.1797	-0.2519	-0.9392	0.0896	11.5113
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4827	2.9226	-0.0648	-0.2454	-0.0095	-3.2689
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4507	3.2460	-0.0582	-0.2215	-0.0218	-13.4003
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6673	6.1053	-0.1680	-0.6255	0.0737	11.6564
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6673	6.1053	-0.1680	-0.6255	0.0737	11.6564
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4683	-0.1519	0.0191	0.0683	-0.0254	-3.1238
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4363	0.1715	0.0258	0.0922	-0.0377	-13.2552

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8964	6.3628	-0.1855	-0.6904	0.0911	15.5860
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8964	6.3628	-0.1855	-0.6904	0.0911	15.5860
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6177	-1.9800	0.0640	0.2348	-0.0411	-4.1211
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5751	-1.5488	0.0729	0.2666	-0.0575	-17.6295

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.5589
Shear X	-1.5030
Shear Z	-0.3800
Moment X	-1.4219
Moment Y (Twist)	0.1687
Moment Z	31.1478

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	9.1797
Shear X	-0.8964
Shear Z	-0.2519
Moment X	-0.9392
Moment Y (Twist)	0.0950
Moment Z	17.6535

Project Details

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

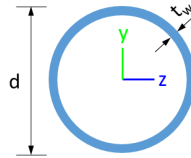


Design Input Information

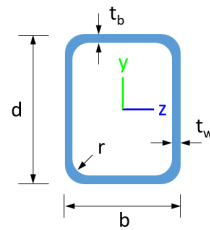
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

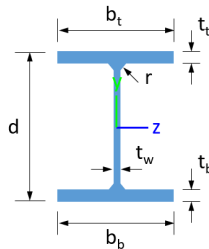
Section Dimensions



ID	Name	d (in)	t_w (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
7	6in Pipe Sch 40	6.63	0.28				



ID	Name	d (in)	b (in)	t_w (in)	t_b (in)	r (in)	
17	HSS5x3x1/4	5.00	3.00	0.23	0.23	0.23	



ID	Name	d (in)	t_w (in)	b_t (in)	b_b (in)	t_t (in)	t_b (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	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)
----	------	----------------------	----------------------	-----------------------------	-----------------------------	--------------------------	-----------------------------	-----------------------------

315	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.09,1.14,1.11,1.11,1.11,1.13,1.11,1.11,1.12,1.12,1.11,1.11,2.69,1.14,1.11,1.11,1.11,1.12	300	200	1
316	20	8.42	8.42	12.95	1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.11,1.12,1.14,1.11,1.11,1.12,1.19,1.11,1.11,1.11,1.12,1.11,1.11,1.12,1.14,1.11,1.11,1.12,1.22	300	200	1

Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	251.16	73.81	42.30	42.30	75.35	75.35
2	251.01	248.88	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	140.46	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	140.46	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	97.43	32.99	6.46	56.26	44.91
14	159.30	97.43	34.34	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	251.16	73.81	42.30	42.30	75.35	75.35
102	251.01	248.88	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	140.46	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	140.46	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	97.43	30.94	6.46	56.26	44.91
114	159.30	97.43	31.61	6.46	56.26	44.91
115	159.30	48.27	14.76	6.46	56.26	44.91
116	159.30	48.27	14.71	6.46	56.26	44.91
201	251.16	73.81	42.30	42.30	75.35	75.35
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95
208	159.30	140.46	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	140.46	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30

212	251.01	240.00	27.10	27.10	75.30	75.30
213	159.30	97.43	30.94	6.46	56.26	44.91
214	159.30	97.43	31.61	6.46	56.26	44.91
215	159.30	48.27	14.22	6.46	56.26	44.91
216	159.30	48.27	14.62	6.46	56.26	44.91
301	251.16	73.81	42.30	42.30	75.35	75.35
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	55.15	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	55.15	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	97.43	32.98	6.46	56.26	44.91
314	159.30	97.43	34.34	6.46	56.26	44.91
315	159.30	48.27	14.49	6.46	56.26	44.91
316	159.30	48.27	14.80	6.46	56.26	44.91

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	0.184	0.736	0.071	0.020	0.005	0.770	#13	0.653	Not Required	Pass
2	0.002	0.369	0.052	0.080	0.009	0.409	#13	0.036	Not Required	Pass
3	0.003	0.613	0.013	0.060	0.005	0.616	#13	0.046	Not Required	Pass
4	0.002	0.521	0.045	0.052	0.011	0.561	#21	0.082	Not Required	Pass
5	0.002	0.379	0.035	0.061	0.009	0.383	#13	0.076	Not Required	Pass
6	0.003	0.740	0.034	0.075	0.007	0.762	#13	0.046	Not Required	Pass
7	0.003	0.460	0.079	0.074	0.020	0.470	#13	0.076	Not Required	Pass
8	0.002	0.095	0.102	0.040	0.007	0.123	#13	0.102	Not Required	Pass
9	0.006	0.083	0.033	0.003	0.002	0.115	#21	0.206	Not Required	Pass
10	0.004	0.605	0.070	0.061	0.014	0.626	#21	0.082	Not Required	Pass
11	0.002	0.099	0.105	0.050	0.007	0.129	#13	0.102	Not Required	Pass
12	0.001	0.477	0.058	0.096	0.011	0.525	#13	0.054	Not Required	Pass
13	0.003	0.205	0.206	0.062	0.009	0.308	#21	0.306	Not Required	Pass
14	0.003	0.168	0.203	0.050	0.009	0.304	#24	0.204	Not Required	Pass
15	0.000	0.059	0.046	0.026	0.004	0.103	#21	Not Required	Not Required	Pass
16	0.000	0.050	0.046	0.022	0.004	0.096	#21	Not Required	Not Required	Pass
101	0.231	0.830	0.009	0.024	0.001	0.918	#13	0.653	Not Required	Pass
102	0.002	0.547	0.069	0.113	0.011	0.600	#21	0.054	Not Required	Pass
103	0.003	0.840	0.019	0.084	0.001	0.853	#13	0.046	Not Required	Pass
104	0.004	0.735	0.076	0.073	0.016	0.781	#21	0.082	Not Required	Pass
105	0.003	0.520	0.081	0.083	0.021	0.534	#13	0.076	Not Required	Pass
106	0.003	0.840	0.018	0.084	0.003	0.843	#13	0.046	Not Required	Pass
107	0.003	0.522	0.069	0.083	0.018	0.533	#13	0.076	Not Required	Pass
108	0.002	0.077	0.085	0.044	0.007	0.162	#21	0.102	Not Required	Pass
109	0.009	0.083	0.023	0.001	0.000	0.110	#21	0.206	Not Required	Pass
110	0.003	0.714	0.068	0.071	0.014	0.759	#21	0.082	Not Required	Pass

111	0.002	0.072	0.086	0.052	0.007	0.159	#21	0.102	Not Required	Pass
112	0.002	0.533	0.070	0.111	0.013	0.589	#13	0.054	Not Required	Pass
113	0.004	0.302	0.214	0.067	0.009	0.483	#21	0.306	Not Required	Pass
114	0.005	0.299	0.214	0.059	0.009	0.481	#21	0.306	Not Required	Pass
115	0.007	0.626	0.112	0.055	0.008	0.708	#21	0.644	Not Required	Pass
116	0.005	0.523	0.111	0.049	0.008	0.622	#21	0.644	Not Required	Pass
201	0.231	0.830	0.009	0.024	0.001	0.918	#13	0.653	Not Required	Pass
202	0.002	0.533	0.070	0.111	0.013	0.589	#13	0.054	Not Required	Pass
203	0.003	0.840	0.018	0.084	0.003	0.843	#13	0.046	Not Required	Pass
204	0.003	0.714	0.068	0.071	0.014	0.759	#21	0.082	Not Required	Pass
205	0.003	0.522	0.069	0.083	0.018	0.533	#13	0.076	Not Required	Pass
206	0.003	0.840	0.019	0.084	0.001	0.853	#13	0.046	Not Required	Pass
207	0.003	0.520	0.081	0.083	0.021	0.534	#13	0.076	Not Required	Pass
208	0.002	0.067	0.111	0.049	0.008	0.178	#21	0.102	Not Required	Pass
209	0.009	0.083	0.023	0.001	0.000	0.110	#21	0.206	Not Required	Pass
210	0.004	0.735	0.076	0.073	0.016	0.781	#21	0.082	Not Required	Pass
211	0.002	0.066	0.112	0.055	0.008	0.169	#21	0.102	Not Required	Pass
212	0.002	0.547	0.069	0.113	0.011	0.600	#21	0.054	Not Required	Pass
213	0.004	0.302	0.214	0.067	0.009	0.483	#21	0.306	Not Required	Pass
214	0.005	0.299	0.214	0.059	0.009	0.481	#21	0.306	Not Required	Pass
215	0.007	0.495	0.099	0.052	0.007	0.563	#21	0.644	Not Required	Pass
216	0.007	0.361	0.099	0.044	0.007	0.459	#21	0.644	Not Required	Pass
301	0.184	0.736	0.071	0.020	0.005	0.770	#13	0.653	Not Required	Pass
302	0.001	0.477	0.058	0.096	0.011	0.525	#13	0.054	Not Required	Pass
303	0.003	0.740	0.034	0.075	0.007	0.762	#13	0.046	Not Required	Pass
304	0.004	0.605	0.070	0.061	0.014	0.626	#21	0.082	Not Required	Pass
305	0.003	0.460	0.079	0.074	0.020	0.470	#13	0.076	Not Required	Pass
306	0.003	0.613	0.013	0.060	0.005	0.616	#13	0.046	Not Required	Pass
307	0.002	0.379	0.035	0.061	0.009	0.383	#13	0.076	Not Required	Pass
308	0.000	0.050	0.046	0.022	0.004	0.096	#21	Not Required	Not Required	Pass
309	0.006	0.083	0.033	0.003	0.002	0.115	#21	0.206	Not Required	Pass
310	0.002	0.521	0.045	0.052	0.011	0.561	#21	0.082	Not Required	Pass
311	0.000	0.059	0.046	0.026	0.004	0.103	#21	Not Required	Not Required	Pass
312	0.002	0.369	0.052	0.080	0.009	0.409	#13	0.036	Not Required	Pass
313	0.003	0.205	0.206	0.062	0.009	0.308	#21	0.204	Not Required	Pass
314	0.003	0.168	0.203	0.050	0.009	0.304	#24	0.306	Not Required	Pass
315	0.007	0.651	0.105	0.050	0.007	0.728	#21	0.644	Not Required	Pass
316	0.005	0.550	0.102	0.040	0.007	0.652	#21	0.644	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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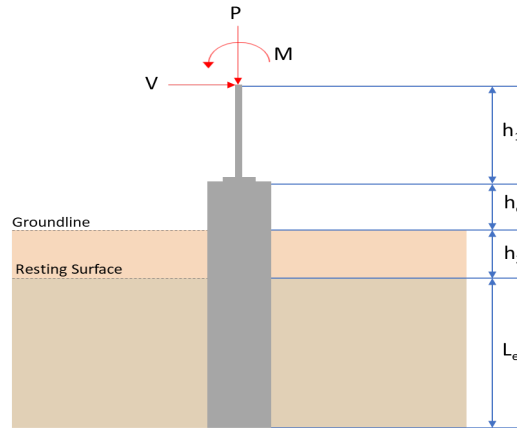
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.180	13.559
V_x (kip)	-0.896	-1.503
V_z (kip)	0.252	0.380
M_x (kipft)	0.939	1.422
M_z (kipft)	17.654	31.147

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,

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

$$H_o = \frac{(-0.896 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.29867 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.654 \text{ kipft}) + ((-0.896 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8847 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.0103 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.252 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.084 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((0.252 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.313 \text{ kipft/ft}$$

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:

$L_{e,z} = 4.1664 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(8.0103 \text{ ft}), (4.1664 \text{ ft})]$$

$$L_{e,req} = 8.01 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$Ratio = \frac{(8.01 \text{ ft})}{(8.5 \text{ ft})}$$

$$Ratio = 0.94235$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.18 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2987 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.2987 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.64935$$

Status: **PASS**
Ratio: **0.650**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.29867 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.8847 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8847 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (5.8847 \text{ kipft/ft})) + (4 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8249 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.8847 \text{ kipft/ft})) + (3 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (5.8847 \text{ kipft/ft})) + (2 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.32868 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.8847 \text{ kipft/ft})) + ((-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2041 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8249 \text{ ft})}{2}$$

$$p_a = 0.43687 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32868 \text{ kip/ft}^2)}{(0.43687 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75236$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2041 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94442$$

Status: **PASS**
Ratio: **0.750**

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

Considering z-direction:

$H_o = 0.084 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.313 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.313 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.084 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.313 \text{ kipft/ft})) + (4 \times (0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.094 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.313 \text{ kipft/ft})) + (3 \times (0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.313 \text{ kipft/ft})) + (2 \times (0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.079347 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.313 \text{ kipft/ft})) + ((0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 0.1748 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.094 \text{ ft})}{2}$$

$$p_a = 0.45705 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.079347 \text{ kip/ft}^2)}{(0.45705 \text{ kip/ft}^2)}$$

(0.1736 kip/ft)

$$\text{Ratio} = 0.17361$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

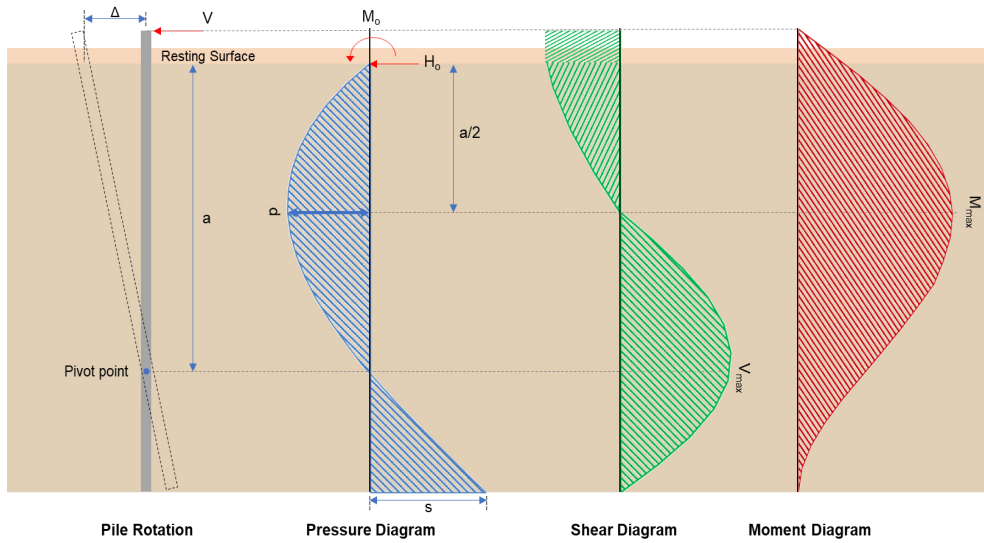
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.1748 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.1371$$

Status: **PASS**
Ratio: **0.170**

Status: **PASS**
Ratio: **0.140**



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.503 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.501 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.147 \text{ kipft}) + ((-1.503 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.382 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.382 \text{ kipft/ft})}{(-0.501 \text{ kip/ft})}$$

$$E = 20.723 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.382 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.501 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (10.382 \text{ kipft/ft})) + (4 \times (-0.501 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8188 \text{ ft}$$

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

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

$$V_{max} = ((-0.501 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (20.723 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8188 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (20.723 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8188 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.4788 \text{ kip}$$

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

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

$$M_{max} = ((-0.501 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(20.723 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (20.723 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (20.723 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 30.62 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.38 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.12667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.422 \text{ kipft}) + ((0.38 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.474 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.474 \text{ kipft/ft})}{(0.12667 \text{ kip/ft})}$$

$$E = 3.7421 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.474 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (0.12667 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.474 \text{ kipft/ft})) + (4 \times (0.12667 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0933 \text{ ft}$$

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

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

$$[\setminus L_e \ / \setminus L_e /]]$$

$$V_{max} = ((0.12667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0933 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0933 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.5497 \text{ kip}$$

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

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

$$M_{max} = ((0.12667 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(3.7421 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.0486 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.559 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.949 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.949 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$(1.8322 \text{ in}^2)$$

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;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.559 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.010813$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.559 \text{ kip} \rightarrow 13559 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13559 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.74 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(186.09 \text{ kip}), (76.74 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.74 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN} [(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.74 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.692 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.4788 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.4788 \text{ kip})}{(74.692 \text{ kip})}$$

$$Ratio = 0.10013$$

Status: **PASS**
Ratio: **0.100**

Considering z-direction:

$V_{max} = 0.5497 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.5497 \text{ kip})}{(74.692 \text{ kip})}$$

$$Ratio = 0.0073597$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 30.62 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(30.62 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.49366$$

Status: **PASS**
Ratio: **0.490**

Considering z-direction:

$M_{max} = 2.0486 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(2.0486 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.033028$$

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

REFERENCES	CALCULATIONS	RESULTS
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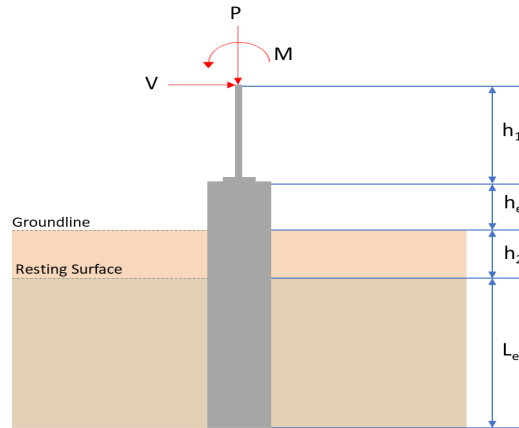
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.5$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	9.180	13.559
V_x (kip)	-0.896	-1.503
V_z (kip)	-0.252	-0.380
M_x (kipft)	-0.939	-1.422
M_z (kipft)	17.654	31.148

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,

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

$$H_o = \frac{(-0.896 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.29867 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.654 \text{ kipft}) + ((-0.896 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 5.8847 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.0103 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.252 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.084 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.939 \text{ kipft}) + ((-0.252 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.313 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.641 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.0103 \text{ ft}), (2.641 \text{ ft})]$$

$$L_{e,req} = 8.01 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.01 \text{ ft})}{(8.5 \text{ ft})}$$

$$\text{Ratio} = 0.94235$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(9.18 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.2987 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.2987 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.64935$$

Status: **PASS**
Ratio: **0.650**

Czeraniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.5 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.8333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.29867 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 5.8847 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.8847 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (5.8847 \text{ kipft/ft})) + (4 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8249 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (5.8847 \text{ kipft/ft})) + (3 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (5.8847 \text{ kipft/ft})) + (2 \times (-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = 0.32868 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (5.8847 \text{ kipft/ft})) + ((-0.29867 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = 1.2041 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(5.8249 \text{ ft})}{2}$$

$$p_a = 0.43687 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32868 \text{ kip/ft}^2)}{(0.43687 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.75236$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2041 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.94442$$

Status: **PASS**
Ratio: **0.750**

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

Considering z-direction:

$H_o = -0.084 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.313 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.313 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.084 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.313 \text{ kipft/ft})) + (4 \times (-0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.094 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.313 \text{ kipft/ft})) + (3 \times (-0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]^2}{(8.5 \text{ ft})^2 \times [(3 \times (0.313 \text{ kipft/ft})) + (2 \times (-0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]}$$

$$p = -0.026411 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.313 \text{ kipft/ft})) + ((-0.084 \text{ kip/ft}) \times (8.5 \text{ ft}))]}{(8.5 \text{ ft})^2}$$

$$s = -0.01148 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.094 \text{ ft})}{2}$$

$$p_a = 0.45705 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.026411 \text{ kip/ft}^2)}{(0.45705 \text{ kip/ft}^2)}$$

$$(0.27100 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.057785$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.5 \text{ ft})$$

$$p_s = 1.275 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

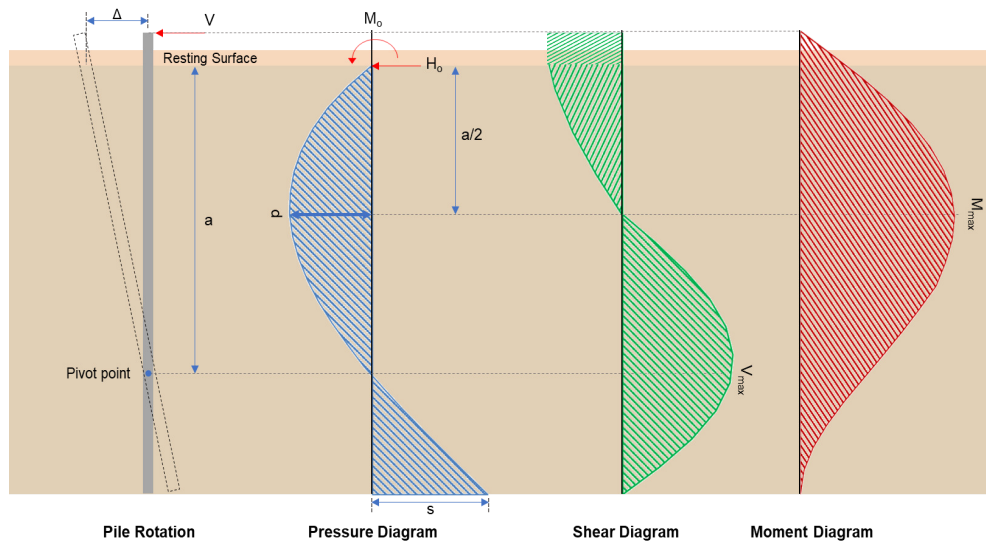
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.01148 \text{ kip/ft}^2)}{(1.275 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0090036$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.503 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.501 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.148 \text{ kipft}) + ((-1.503 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 10.383 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(10.383 \text{ kipft/ft})}{(-0.501 \text{ kip/ft})}$$

$$E = 20.724 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (10.383 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.501 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (10.383 \text{ kipft/ft})) + (4 \times (-0.501 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 5.8188 \text{ ft}$$

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

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

$$V_{max} = ((-0.501 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (20.724 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8188 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (20.724 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8188 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.479 \text{ kip}$$

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

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

$$M_{max} = ((-0.501 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(20.724 \text{ ft})}{(8.5 \text{ ft})} + \frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (20.724 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (20.724 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(5.8188 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 30.621 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.38 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.12667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.422 \text{ kipft}) + ((-0.38 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.474 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.474 \text{ kipft/ft})}{(-0.12667 \text{ kip/ft})}$$

$$E = 3.7421 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.474 \text{ kipft/ft}) \times (8.5 \text{ ft})) + (3 \times (-0.12667 \text{ kip/ft}) \times (8.5 \text{ ft})^2)}{(6 \times (0.474 \text{ kipft/ft})) + (4 \times (-0.12667 \text{ kip/ft}) \times (8.5 \text{ ft}))}$$

$$a = 6.0933 \text{ ft}$$

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

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

$$[\setminus L_e \ / \setminus L_e /]]$$

$$V_{max} = ((-0.12667 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0933 \text{ ft})}{(8.5 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0933 \text{ ft})}{(8.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.5497 \text{ kip}$$

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

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

$$M_{max} = ((-0.12667 \text{ kip/ft}) \times (36 \text{ in}) \times (8.5 \text{ ft})) \times \left[\left(\frac{(3.7421 \text{ ft})}{(8.5 \text{ ft})} + \frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 3 \right) \times \left(\frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7421 \text{ ft})}{(8.5 \text{ ft})} + 2 \right) \times \left(\frac{(6.0933 \text{ ft})}{2 \times (8.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.0486 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.559 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.949 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.949 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

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;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.559 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.010813$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.559 \text{ kip} \rightarrow 13559 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(13559 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 76.74 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (76.74 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 76.74 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((76.74 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 74.692 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.479 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.479 \text{ kip})}{(74.692 \text{ kip})}$$

$$Ratio = 0.10013$$

Status: **PASS**
Ratio: **0.100**

Considering z-direction:

$V_{max} = 0.5497 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.5497 \text{ kip})}{(74.692 \text{ kip})}$$

$$Ratio = 0.0073597$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 30.621 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(30.621 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.49367$$

Status: **PASS**
Ratio: **0.490**

Considering z-direction:

$M_{max} = 2.0486 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(2.0486 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.033028$$

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

REFERENCES	CALCULATIONS	RESULTS
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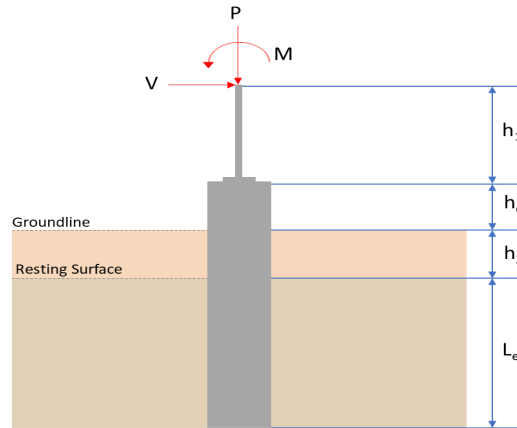
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.535	17.077
V_x (kip)	-1.071	-1.784
V_z (kip)	-0.032	-0.049
M_x (kipft)	-0.121	-0.188
M_z (kipft)	19.904	35.114

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,

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

$$H_o = \frac{(-1.071 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.357 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.904 \text{ kipft}) + ((-1.071 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.6347 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2273 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.032 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.010667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.121 \text{ kipft}) + ((-0.032 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.040333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5235 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.2273 \text{ ft}), (1.5235 \text{ ft})]$$

$$L_{e,req} = 8.227 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.227 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.94023$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.535 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.6319 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.6319 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.81593$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.9167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.357 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.6347 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6347 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (6.6347 \text{ kipft/ft})) + (4 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0075 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.6347 \text{ kipft/ft})) + (3 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (6.6347 \text{ kipft/ft})) + (2 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.33205 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.6347 \text{ kipft/ft})) + ((-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2489 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0075 \text{ ft})}{2}$$

$$p_a = 0.45056 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.33205 \text{ kip/ft}^2)}{(0.45056 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73696$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2489 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95157$$

Status: **PASS**
Ratio: **0.740**

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = -0.010667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.040333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.040333 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.010667 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.040333 \text{ kipft/ft})) + (4 \times (-0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2757 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.040333 \text{ kipft/ft})) + (3 \times (-0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.040333 \text{ kipft/ft})) + (2 \times (-0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = -0.0032995 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.040333 \text{ kipft/ft})) + ((-0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = -0.0015593 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2757 \text{ ft})}{2}$$

$$p_a = 0.47068 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0032995 \text{ kip/ft}^2)}{(0.47068 \text{ kip/ft}^2)}$$

$$(0.0000 \text{ kip/ft}^2)$$

$$\text{Ratio} = -0.00701$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

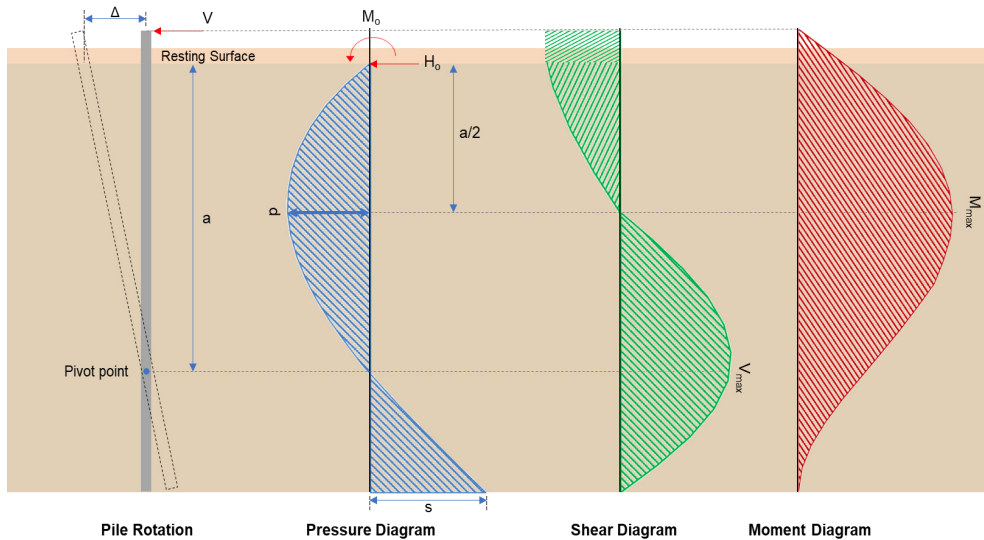
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(-0.0015593 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.001188$$

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,

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

$$H_o = \frac{(-1.784 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.59467 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.114 \text{ kipft}) + ((-1.784 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 11.705 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.705 \text{ kipft/ft})}{(-0.59467 \text{ kip/ft})}$$

$$E = 19.683 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.705 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.59467 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.705 \text{ kipft/ft})) + (4 \times (-0.59467 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

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

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

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

$$V_{max} = ((-0.59467 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2804 \text{ kip}$$

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

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

$$M_{max} = ((-0.59467 \text{ kip/ft}) \times (36 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(19.683 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.805 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(-0.049 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.016333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.188 \text{ kipft}) + ((-0.049 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.062667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.062667 \text{ kipft/ft})}{(-0.016333 \text{ kip/ft})}$$

$$E = 3.8367 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.062667 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.016333 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.062667 \text{ kipft/ft})) + (4 \times (-0.016333 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2732 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \right]$$

$$V_{max} = ((-0.016333 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.2732 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.2732 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.070732 \text{ kip}$$

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

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

$$M_{max} = ((-0.016333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(3.8367 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.27128 \text{ kipft}$$

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,

Table 22.4.2.1

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - Gross area of concrete,

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{(17.077 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2)), \frac{(17.077 \text{ kip})}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.839 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.839 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\quad}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10\emptyset: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(17.077 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.013619$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.077 \text{ kip} \rightarrow 17077 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(17077 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.337 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (77.337 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.337 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.337 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 75.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.2804 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.2804 \text{ kip})}{(75.08 \text{ kip})}$$

$$Ratio = 0.11029$$

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

Considering z-direction:

$V_{max} = 0.070732 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.070732 \text{ kip})}{(75.08 \text{ kip})}$$

$$Ratio = 0.00094209$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b

$\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.805 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.805 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.56112$$

Status: **PASS**
Ratio: **0.560**

Considering z-direction:

$M_{max} = 0.27128 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{M_u}{\phi M_n}$$

$$Ratio = \frac{(0.27128 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0043735$$

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

REFERENCES	CALCULATIONS	RESULTS
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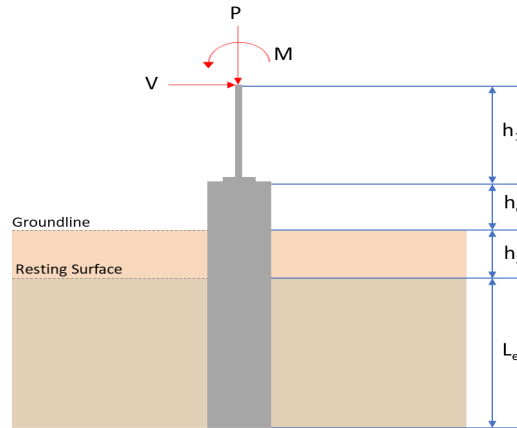
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: round

$D = 36$ in - Pile diameter

$L = 8.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.535	17.077
V_x (kip)	-1.071	-1.784
V_z (kip)	0.032	0.049
M_x (kipft)	0.121	0.188
M_z (kipft)	19.904	35.115

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,

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

$$H_o = \frac{(-1.071 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.357 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.904 \text{ kipft}) + ((-1.071 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 6.6347 \text{ kipft/ft}$$

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:

$L_{e,x} = 8.2273 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.032 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.010667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.121 \text{ kipft}) + ((0.032 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.040333 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9121 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(8.2273 \text{ ft}), (1.9121 \text{ ft})]$$

$$L_{e,req} = 8.227 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (8.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 8.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(8.227 \text{ ft})}{(8.75 \text{ ft})}$$

$$\text{Ratio} = 0.94023$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = \pi \left(\frac{D}{2}\right)^2$$

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(11.535 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 1.6319 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.6319 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.81593$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(8.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.9167$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.357 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 6.6347 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.6347 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (6.6347 \text{ kipft/ft})) + (4 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.0075 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (6.6347 \text{ kipft/ft})) + (3 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (6.6347 \text{ kipft/ft})) + (2 \times (-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.33205 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (6.6347 \text{ kipft/ft})) + ((-0.357 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 1.2489 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.0075 \text{ ft})}{2}$$

$$p_a = 0.45056 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.33205 \text{ kip/ft}^2)}{(0.45056 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73696$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2489 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95157$$

Status: **PASS**
Ratio: **0.740**

Status: **PASS**
Ratio: **0.950**

Considering z-direction:

$H_o = 0.010667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.040333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.040333 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.010667 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.040333 \text{ kipft/ft})) + (4 \times (0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2757 \text{ ft}$$

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

$$p = \frac{1.178 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$$

$$p = \frac{1.178 \times [(4 \times (0.040333 \text{ kipft/ft})) + (3 \times (0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]^2}{(8.75 \text{ ft})^2 \times [(3 \times (0.040333 \text{ kipft/ft})) + (2 \times (0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]}$$

$$p = 0.0097405 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

$$s = \frac{9.425 [(2 M_o) + (H_o L_e)]}{L_e^2}$$

$$s = \frac{9.425 \times [(2 \times (0.040333 \text{ kipft/ft})) + ((0.010667 \text{ kip/ft}) \times (8.75 \text{ ft}))]}{(8.75 \text{ ft})^2}$$

$$s = 0.02142 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(6.2757 \text{ ft})}{2}$$

$$p_a = 0.47068 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0097405 \text{ kip/ft}^2)}{(0.47068 \text{ kip/ft}^2)}$$

$$(0.020695 \text{ kip/ft}^2)$$

$$\text{Ratio} = 0.020695$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (8.75 \text{ ft})$$

$$p_s = 1.3125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

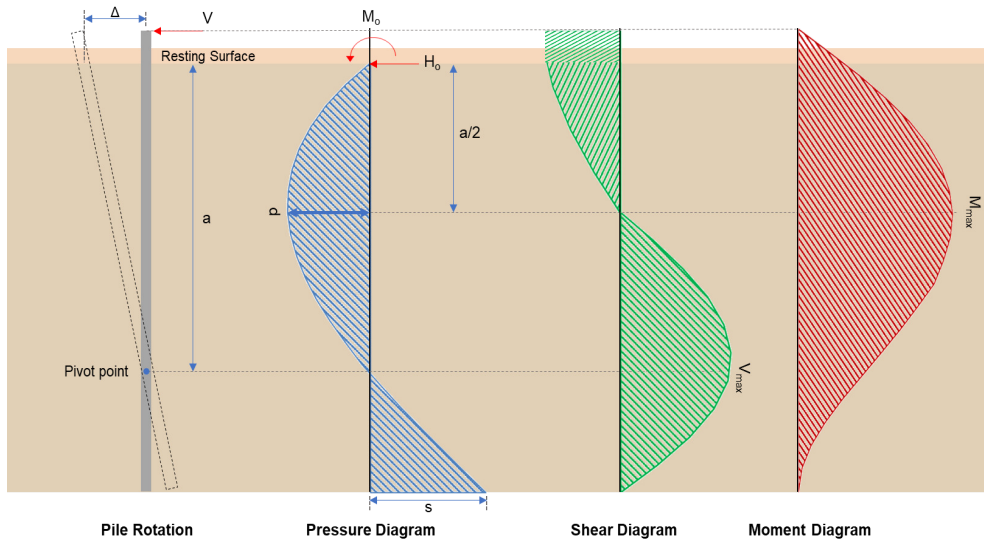
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.02142 \text{ kip/ft}^2)}{(1.3125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.01632$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.784 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.59467 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.115 \text{ kipft}) + ((-1.784 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 11.705 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(11.705 \text{ kipft/ft})}{(-0.59467 \text{ kip/ft})}$$

$$E = 19.683 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (11.705 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (-0.59467 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (11.705 \text{ kipft/ft})) + (4 \times (-0.59467 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

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

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

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

$$V_{max} = ((-0.59467 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2806 \text{ kip}$$

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

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

$$M_{max} = ((-0.59467 \text{ kip/ft}) \times (36 \text{ in}) \times (8.75 \text{ ft})) \times \left[\left(\frac{(19.683 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.683 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 34.806 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

$$H_o = \frac{V_z}{D}$$

$$H_o = \frac{(0.049 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.016333 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.188 \text{ kipft}) + ((0.049 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.062667 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.062667 \text{ kipft/ft})}{(0.016333 \text{ kip/ft})}$$

$$E = 3.8367 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.062667 \text{ kipft/ft}) \times (8.75 \text{ ft})) + (3 \times (0.016333 \text{ kip/ft}) \times (8.75 \text{ ft})^2)}{(6 \times (0.062667 \text{ kipft/ft})) + (4 \times (0.016333 \text{ kip/ft}) \times (8.75 \text{ ft}))}$$

$$a = 6.2732 \text{ ft}$$

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

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

$$\left[\frac{L_e}{L_e} \quad / \quad \frac{L_e}{L_e} \right]$$

$$V_{max} = \left((0.016333 \text{ kip/ft}) \times (36 \text{ in}) \right) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.2732 \text{ ft})}{(8.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.2732 \text{ ft})}{(8.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.070732 \text{ kip}$$

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

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

$$M_{max} = \left((0.016333 \text{ kip/ft}) \times (36 \text{ in}) \times (8.75 \text{ ft}) \right) \times \left[\left(\frac{(3.8367 \text{ ft})}{(8.75 \text{ ft})} + \frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 3 \right) \times \left(\frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.8367 \text{ ft})}{(8.75 \text{ ft})} + 2 \right) \times \left(\frac{(6.2732 \text{ ft})}{2 \times (8.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.27128 \text{ kipft}$$

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,

$\alpha = 0.85$ - Alpha factor for axial strength,

$A_g = 1017.9 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(17.077 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -36.839 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-36.839 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$= \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = \frac{\lambda}{(1.8408 \text{ in}^2)}$</p> <p style="text-align: center;">$Ratio = 0.99533$</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 style="text-align: center;">$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10Ø: Use #3(0.375 in)</p> <p>$s_{ties} = Max [16 d_{bar}, (48 d_{ties}), D]$</p> <p>$s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$</p> <p style="text-align: center;">$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 6 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 1.000</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;">$\phi P_N = \phi 0.85 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 1253.9 \text{ kip}$</p> <p><i>Ratio - Capacity</i></p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(17.077 \text{ kip})}{(1253.9 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.013619$</p>	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 36 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (36 \text{ in})$</p> <p style="text-align: center;">$d = 28.8 \text{ in}$</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;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.71796$</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;">$V_{c,max} = 5 \times (0.71796) \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$</p>	

$$V_{c,max} = 186.09 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 17.077 \text{ kip} \rightarrow 17077 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + \frac{(17077 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 77.337 \text{ kip}$$

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)

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 204.04 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(186.09 \text{ kip}), (77.337 \text{ kip}), (204.04 \text{ kip})]$$

$$V_c = 77.337 \text{ kip}$$

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)

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

$$V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{s,a} = 414.72 \text{ kip}$$

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)

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

$$V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$$

$$V_{s,b} = 38.17 \text{ kip}$$

V_s - Governing shear strength of steel

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

$$V_s = \text{MIN}[(414.72 \text{ kip}), (38.17 \text{ kip})]$$

$$V_s = 38.17 \text{ kip}$$

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((77.337 \text{ kip}) + (38.17 \text{ kip}))$$

$$\phi V_n = 75.08 \text{ kip}$$

Considering x-direction:

$V_{max} = 8.2806 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(8.2806 \text{ kip})}{(75.08 \text{ kip})}$$

$$Ratio = 0.11029$$

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

Considering z-direction:

$V_{max} = 0.070732 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.070732 \text{ kip})}{(75.08 \text{ kip})}$$

$$Ratio = 0.00094209$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

$$S_m = \frac{\pi D^3}{32}$$

$$S_m = \frac{\pi \times (36 \text{ in})^3}{32}$$

$$S_m = 4580.4 \text{ in}^3$$

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

$$\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(2.5 \text{ ksi})} \times 4580.442 \text{ in}^3$$

$$\phi M_{n,1} = 62.027 \text{ kipft}$$

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 527.23 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(62.027 \text{ kipft}), (527.23 \text{ kipft})]$$

$$\phi M_n = 62.027 \text{ kipft}$$

Considering x-direction:

$M_{max} = 34.806 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(34.806 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.56114$$

Status: **PASS**
Ratio: **0.560**

Considering z-direction:

$M_{max} = 0.27128 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$ratio = \frac{1}{\phi M_n}$$

$$Ratio = \frac{(0.27128 \text{ kipft})}{(62.027 \text{ kipft})}$$

$$Ratio = 0.0043735$$

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