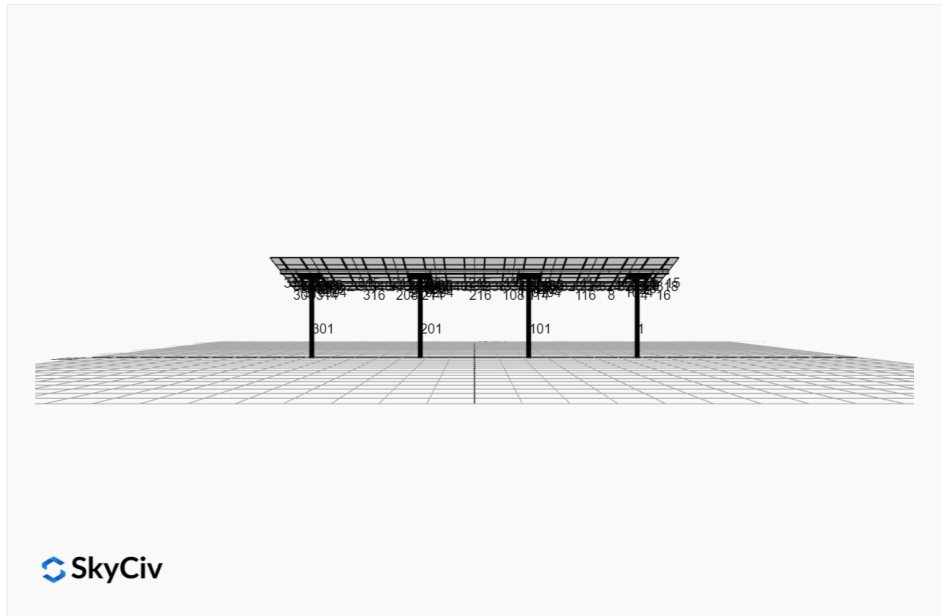


Project Details



Project Name: LFRC **Date:** Wed Oct 23 2024
Location: 202 S 2nd St, Livingston, MT 59047, USA **Number of Modules:** 50
Unique ID: 4P-19.75-8TOP-XD-12-L-5Hx10W-BBLA **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



Array Dimensions N/S	18.79 ft
Array Dimensions E/W	69.67 ft
Winter Tilt Angle	15
Front Edge Clearance	12 ft

MT Solar Bill of Materials (4P-19.75-8TOP-XD-12-L-5Hx10W-BBLA)

Part	Short Description	BOM Qty
MTS-PC-8	8IN Pole Cap Assembly	4
MTS-HF-XD	H-Frame Assembly-XD	4
MTS-XD-Wing-12	12IN XD Wing	4
MTS-XD-Splice-90	90IN XD Splice	6
MTS-XD-Splice-57	57IN XD Splice	6
MTS-CLAMP-ANGLE-4PK	Angle Clamp	10

Rail Bill of Materials

Part	Qty
Rails (223in)	20
Rail Attachment	80
Module Mid Clamp	80
Module End Clamp	40
Ground Lug	10

Site Details:



Site Address: 202 S 2nd St, Livingston, MT 59047, USA

Array Specification

Duty Classification:	XD
Module Width:	44.60 in
Module Length:	82.60in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Winter Tilt Angle:	15
Front Edge Clearance:	12
Total Array Height at Tilt:	16.86 ft
Total Frame Length:	68.75 ft
Frame Weight:	5447 lbs
Array Dimensions N/S:	18.79 ft
Array Dimensions E/W:	69.67 ft
Rail Length:	225.50 in
Rail Spacing:	3.48 ft

Support Specifications

Pole Size:	8in Pipe Sch 40
Pole Length above Grade:	14.43 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 5.75 ft
Foundation Volume:	14.222 y ³

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sand
Site Location:	202 S 2nd St, Livingston, MT 59047, USA
Wind Speed:	101 mph

Snow Load:

33 psf

Design Disclaimer

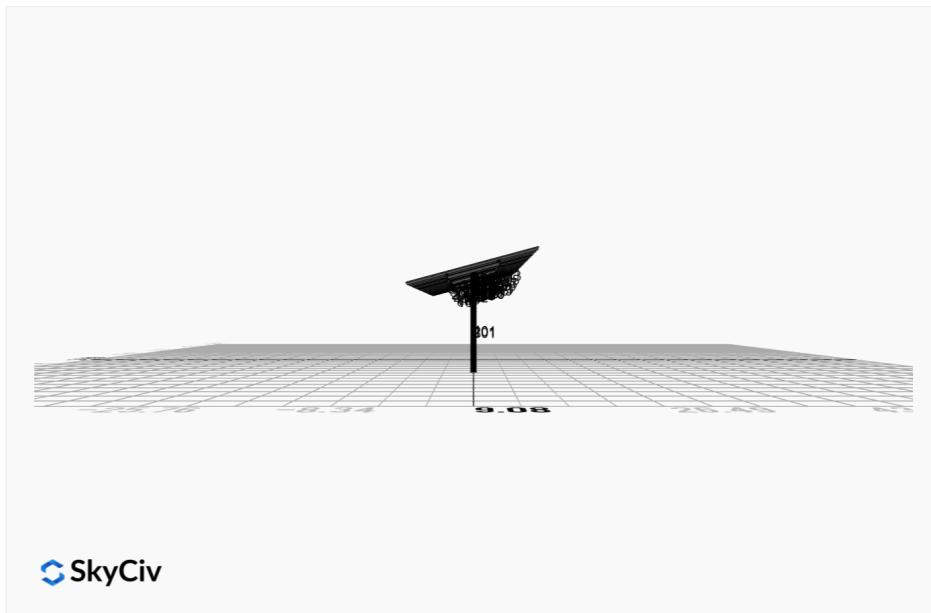
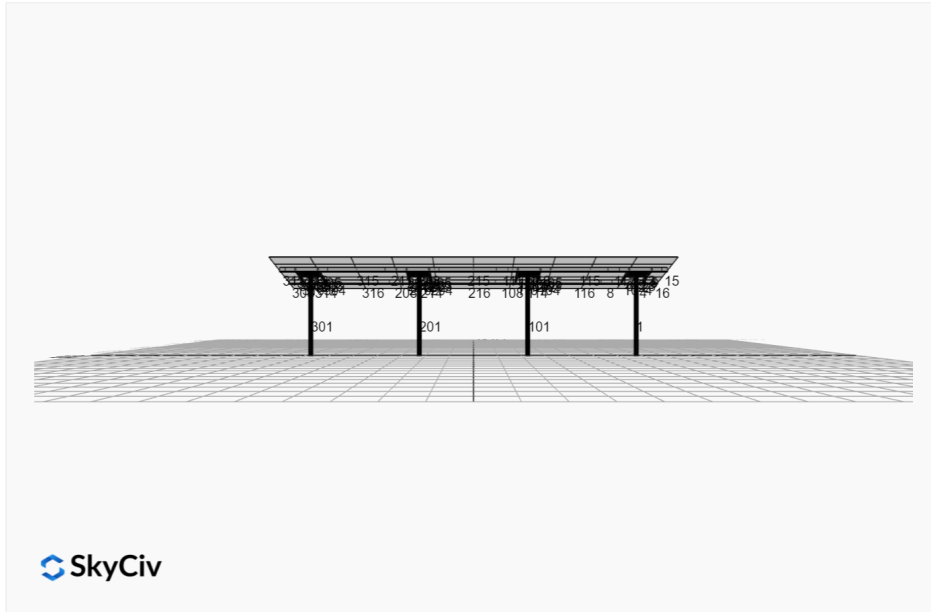
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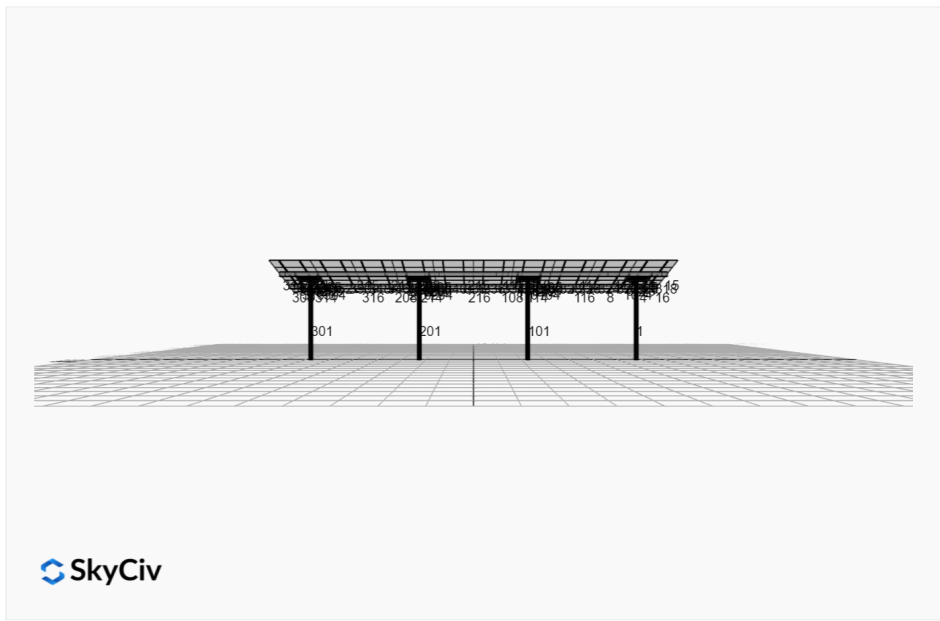
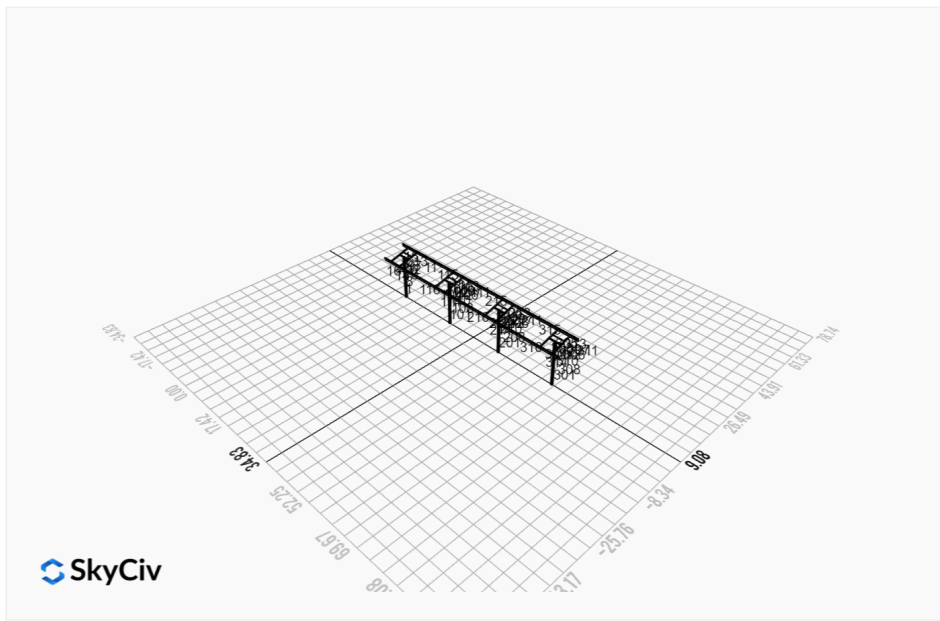
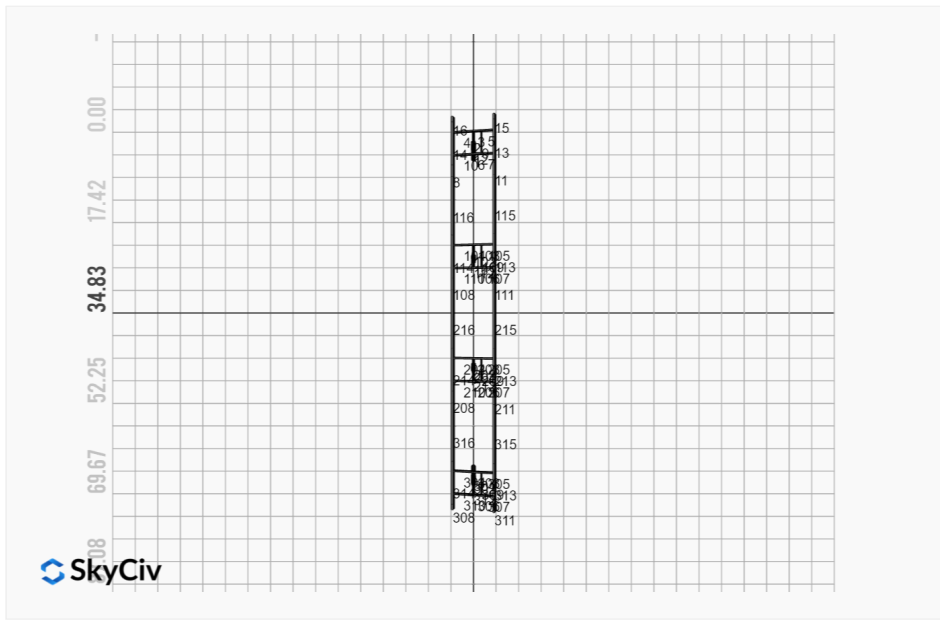
AutoDesigner Input

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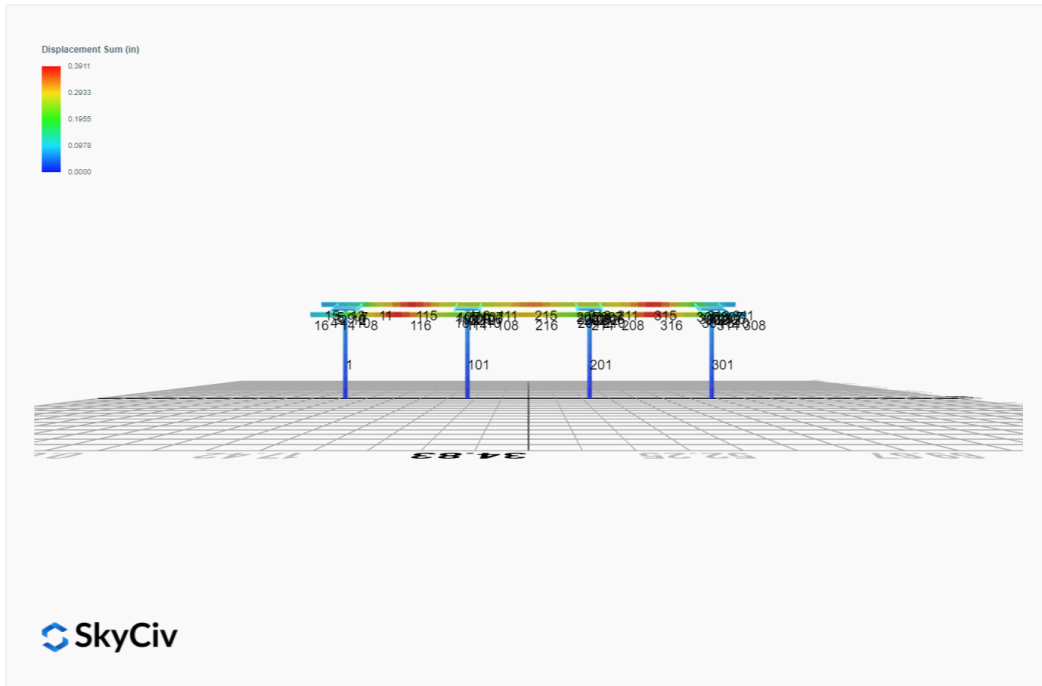
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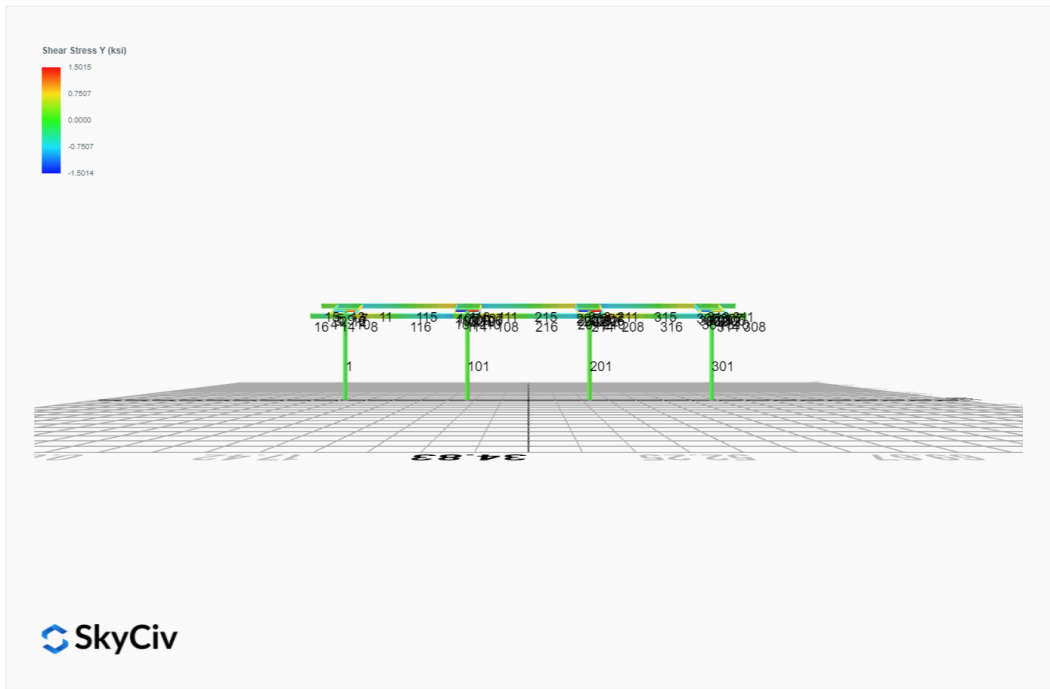
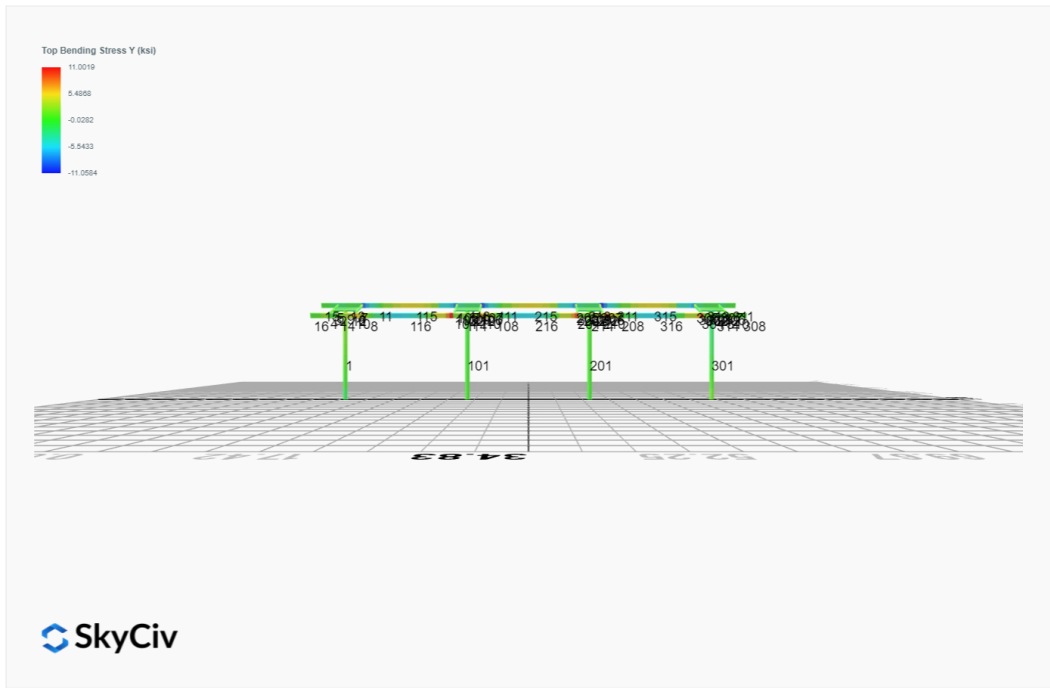
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

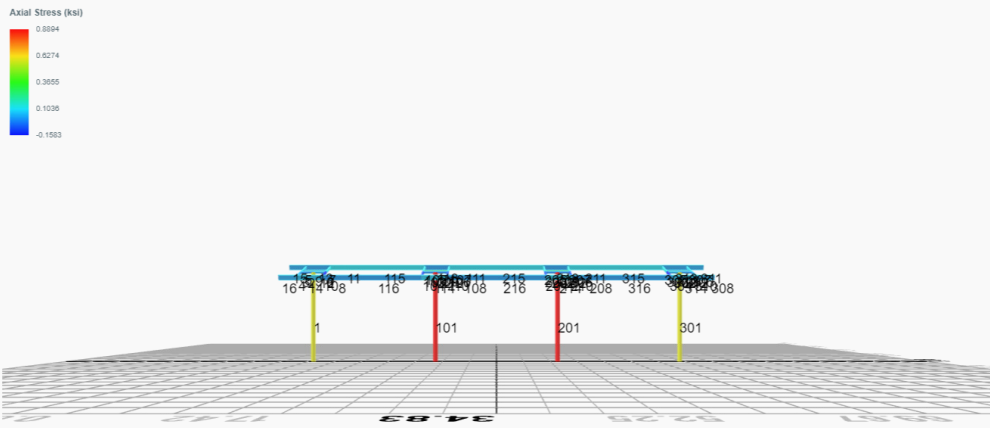




FEM Results (Envelope Worst Case for each member)







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	0.0197	2.1825	0.0753	0.3433	-0.0291	-0.2267
ULS: 2. D + L	0.0197	2.1825	0.0753	0.3433	-0.0291	-0.2267
ULS: 3. D + (S or Lr or R)	0.0847	7.1657	0.3270	1.4922	-0.1268	-1.0810
ULS: 3. D + (S or Lr or R)	0.0197	2.1825	0.0753	0.3433	-0.0291	-0.2267
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0684	5.9199	0.2641	1.2050	-0.1024	-0.8674
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0197	2.1825	0.0753	0.3433	-0.0291	-0.2267
ULS: 5b. D + 0.7E	0.0197	2.1825	0.0753	0.3433	-0.0291	-0.2267
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0684	5.9199	0.2641	1.2050	-0.1024	-0.8674
ULS: 8. 0.6D + 0.7E	0.0118	1.3095	0.0452	0.2060	-0.0174	-0.1360
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8373	5.1867	0.2556	1.1560	-0.2625	13.9823
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8373	5.1867	0.2556	1.1560	-0.2625	13.9823
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6522	-0.0871	-0.0506	-0.2231	0.1270	-7.6917
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6146	0.1975	-0.0612	-0.2693	0.1622	-16.6417
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5743	8.1730	0.3992	1.8145	-0.2774	9.7894
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5743	8.1730	0.3992	1.8145	-0.2774	9.7894
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5429	4.2177	0.1696	0.7801	0.0146	-6.4662
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5147	4.4312	0.1617	0.7455	0.0411	-13.1787
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6231	4.4356	0.2105	0.9528	-0.2041	10.4301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6231	4.4356	0.2105	0.9528	-0.2041	10.4301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4941	0.4803	-0.0191	-0.0815	0.0880	-5.8255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4659	0.6937	-0.0271	-0.1162	0.1144	-12.5380
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8452	4.3137	0.2254	1.0187	-0.2508	14.0730
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8452	4.3137	0.2254	1.0187	-0.2508	14.0730
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6444	-0.9601	-0.0808	-0.3605	0.1386	-7.6011
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6068	-0.6755	-0.0913	-0.4067	0.1738	-16.5511

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	13.0984
Shear X	-1.4283
Shear Z	0.6490
Moment X	2.9642
Moment Y (Twist)	0.4802
Moment Z	29.0202

Worst Case Reactions ASD

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

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

Result	Value (kip, kip-ft)
Axial	8.1730
Shear X	-0.8452
Shear Z	0.3992
Moment X	1.8145
Moment Y (Twist)	0.2774
Moment Z	16.6417

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	-0.0197	2.9302	-0.0084	-0.0388	0.0070	0.2925
ULS: 2. D + L	-0.0197	2.9302	-0.0084	-0.0388	0.0070	0.2925
ULS: 3. D + (S or Lr or R)	-0.0847	10.4001	-0.0365	-0.1682	0.0298	1.2009
ULS: 3. D + (S or Lr or R)	-0.0197	2.9302	-0.0084	-0.0388	0.0070	0.2925
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0684	8.5326	-0.0295	-0.1359	0.0241	0.9738

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0197	2.9302	-0.0084	-0.0388	0.0070	0.2925
ULS: 5b. D + 0.7E	-0.0197	2.9302	-0.0084	-0.0388	0.0070	0.2925
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0684	8.5326	-0.0295	-0.1359	0.0241	0.9738
ULS: 8. 0.6D + 0.7E	-0.0118	1.7581	-0.0051	-0.0233	0.0042	0.1755
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1686	7.4122	-0.0118	-0.0569	-0.0196	19.0969
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.1686	7.4122	-0.0118	-0.0569	-0.0196	19.0969
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8695	-0.4793	-0.0000	0.0007	0.0119	-9.8356
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6996	0.0105	-0.0164	-0.0718	0.0519	-20.3785
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9301	11.8941	-0.0320	-0.1494	0.0042	15.0771
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9301	11.8941	-0.0320	-0.1494	0.0042	15.0771
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5984	5.9755	-0.0232	-0.1062	0.0278	-6.6222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4710	6.3428	-0.0354	-0.1606	0.0579	-14.5295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8813	6.2917	-0.0110	-0.0524	-0.0130	14.3958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8813	6.2917	-0.0110	-0.0524	-0.0130	14.3958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6472	0.3731	-0.0021	-0.0092	0.0106	-7.3035
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5198	0.7404	-0.0144	-0.0636	0.0407	-15.2108
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1607	6.2401	-0.0085	-0.0414	-0.0224	18.9799
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.1607	6.2401	-0.0085	-0.0414	-0.0224	18.9799
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8774	-1.6514	0.0034	0.0162	0.0091	-9.9526
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7075	-1.1616	-0.0130	-0.0563	0.0492	-20.4955

Worst Case Reactions LFRD

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	19.2003
Shear X	-1.9650
Shear Z	-0.0649
Moment X	-0.2963
Moment Y (Twist)	0.1010
Moment Z	35.0561

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	11.8941
Shear X	-1.1686
Shear Z	-0.0365
Moment X	-0.1682
Moment Y (Twist)	0.0579
Moment Z	20.4955

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	-0.0197	2.9302	0.0084	0.0388	-0.0070	0.2925
ULS: 2. D + L	-0.0197	2.9302	0.0084	0.0388	-0.0070	0.2925
ULS: 3. D + (S or Lr or R)	-0.0847	10.4001	0.0365	0.1682	-0.0297	1.2009
ULS: 3. D + (S or Lr or R)	-0.0197	2.9302	0.0084	0.0388	-0.0070	0.2925
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0684	8.5326	0.0295	0.1358	-0.0240	0.9738
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0197	2.9302	0.0084	0.0388	-0.0070	0.2925
ULS: 5b. D + 0.7E	-0.0197	2.9302	0.0084	0.0388	-0.0070	0.2925
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0684	8.5326	0.0295	0.1358	-0.0240	0.9738
ULS: 8. 0.6D + 0.7E	-0.0118	1.7581	0.0051	0.0233	-0.0042	0.1755
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.1686	7.4122	0.0118	0.0569	0.0196	19.0969
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.1686	7.4122	0.0118	0.0569	0.0196	19.0969
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.8695	-0.4793	0.0000	-0.0007	-0.0119	-9.8356
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6996	0.0105	0.0164	0.0718	-0.0519	-20.3785

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	-0.9301	11.8941	0.0320	0.1494	-0.0041	15.0771
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9301	11.8941	0.0320	0.1494	-0.0041	15.0771
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5984	5.9755	0.0232	0.1062	-0.0277	-6.6222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4710	6.3428	0.0354	0.1606	-0.0577	-14.5295
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.8813	6.2917	0.0110	0.0524	0.0130	14.3958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.8813	6.2917	0.0110	0.0524	0.0130	14.3958
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.6472	0.3731	0.0021	0.0092	-0.0106	-7.3035
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5198	0.7404	0.0144	0.0636	-0.0407	-15.2108
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.1607	6.2401	0.0085	0.0414	0.0224	18.9799
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.1607	6.2401	0.0085	0.0414	0.0224	18.9799
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.8774	-1.6514	-0.0034	-0.0162	-0.0091	-9.9526
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.7075	-1.1616	0.0130	0.0563	-0.0491	-20.4955

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	19.2003
Shear X	-1.9650
Shear Z	0.0649
Moment X	0.2965
Moment Y (Twist)	0.1009
Moment Z	35.0562

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	11.8941
Shear X	-1.1686
Shear Z	0.0365
Moment X	0.1682
Moment Y (Twist)	0.0577
Moment Z	20.4955

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	0.0197	2.1825	-0.0753	-0.3433	0.0291	-0.2267
ULS: 2. D + L	0.0197	2.1825	-0.0753	-0.3433	0.0291	-0.2267
ULS: 3. D + (S or Lr or R)	0.0847	7.1657	-0.3270	-1.4922	0.1269	-1.0808
ULS: 3. D + (S or Lr or R)	0.0197	2.1825	-0.0753	-0.3433	0.0291	-0.2267
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0684	5.9199	-0.2641	-1.2050	0.1025	-0.8673
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0197	2.1825	-0.0753	-0.3433	0.0291	-0.2267
ULS: 5b. D + 0.7E	0.0197	2.1825	-0.0753	-0.3433	0.0291	-0.2267
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0684	5.9199	-0.2641	-1.2050	0.1025	-0.8673
ULS: 8. 0.6D + 0.7E	0.0118	1.3095	-0.0452	-0.2060	0.0174	-0.1360
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8373	5.1867	-0.2556	-1.1560	0.2625	13.9823
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8373	5.1867	-0.2556	-1.1560	0.2625	13.9823
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.6522	-0.0871	0.0506	0.2231	-0.1269	-7.6917
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.6146	0.1975	0.0612	0.2693	-0.1622	-16.6417
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.5743	8.1730	-0.3993	-1.8145	0.2775	9.7895
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.5743	8.1730	-0.3993	-1.8145	0.2775	9.7895
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.5429	4.2177	-0.1696	-0.7802	-0.0145	-6.4661
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.5147	4.4312	-0.1617	-0.7455	-0.0410	-13.1786
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6231	4.4356	-0.2105	-0.9528	0.2041	10.4301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6231	4.4356	-0.2105	-0.9528	0.2041	10.4301
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.4941	0.4803	0.0191	0.0815	-0.0879	-5.8255
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.4659	0.6937	0.0271	0.1162	-0.1144	-12.5380

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8452	4.3137	-0.2254	-1.0187	0.2508	14.0730
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8452	4.3137	-0.2254	-1.0187	0.2508	14.0730
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.6444	-0.9601	0.0808	0.3605	-0.1386	-7.6011
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.6068	-0.6755	0.0913	0.4067	-0.1738	-16.5511

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	13.0984
Shear X	-1.4283
Shear Z	-0.6490
Moment X	-2.9644
Moment Y (Twist)	0.4805
Moment Z	29.0205

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.1730
Shear X	-0.8452
Shear Z	-0.3993
Moment X	-1.8145
Moment Y (Twist)	0.2775
Moment Z	16.6417

Project Details

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

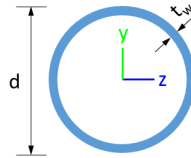


Design Input Information

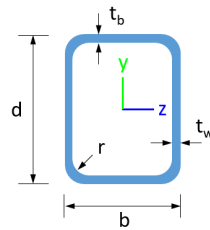
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

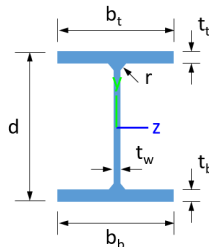
Section Dimensions



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				
9	8in Pipe Sch 40	8.63	0.32				



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	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.12,1.08,1.08,1.07,1.09,1.08,1.08,1.08,1.08,1.08,1.15,1.11,1.08,1.08,1.07,1.09	300	200	1
316	20	6.63	6.63	10.20	1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.11,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.08,1.13	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	377.97	123.81	83.29	83.29	113.39	113.39
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	40.65	6.46	56.26	44.91
14	159.30	97.43	46.90	6.46	56.26	44.91
15	159.30	137.23	46.90	6.46	56.26	44.91
16	159.30	137.23	46.90	6.46	56.26	44.91
101	377.97	123.81	83.29	83.29	113.39	113.39
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	32.10	6.46	56.26	44.91
114	159.30	97.43	32.32	6.46	56.26	44.91
115	159.30	75.13	20.56	6.46	56.26	44.91
116	159.30	75.13	20.63	6.46	56.26	44.91
201	377.97	123.81	83.29	83.29	113.39	113.39
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	248.88	27.10	27.10	75.30	75.30
213	159.30	97.43	32.10	6.46	56.26	44.91
214	159.30	97.43	32.32	6.46	56.26	44.91
215	159.30	75.13	20.90	6.46	56.26	44.91
216	159.30	75.13	20.99	6.46	56.26	44.91
301	377.97	123.81	83.29	83.29	113.39	113.39
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	137.23	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	137.23	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	40.65	6.46	56.26	44.91
314	159.30	97.43	46.90	6.46	56.26	44.91
315	159.30	75.13	20.67	6.46	56.26	44.91
316	159.30	75.13	20.71	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	0.106	0.348	0.077	0.012	0.006	0.356	#16	0.619	Not Required	Pass
2	0.002	0.273	0.044	0.066	0.008	0.308	#21	0.054	Not Required	Pass
3	0.004	0.469	0.012	0.045	0.006	0.475	#21	0.046	Not Required	Pass
4	0.002	0.445	0.046	0.045	0.010	0.492	#21	0.122	Not Required	Pass
5	0.003	0.291	0.012	0.047	0.003	0.298	#21	0.076	Not Required	Pass
6	0.006	0.698	0.074	0.071	0.021	0.775	#21	0.046	Not Required	Pass
7	0.007	0.432	0.113	0.069	0.029	0.459	#21	0.076	Not Required	Pass
8	0.002	0.157	0.117	0.039	0.013	0.226	#21	0.102	Not Required	Pass
9	0.004	0.084	0.060	0.004	0.004	0.145	#21	0.206	Not Required	Pass
10	0.007	0.645	0.095	0.065	0.020	0.679	#21	0.082	Not Required	Pass
11	0.005	0.158	0.123	0.043	0.013	0.224	#21	0.102	Not Required	Pass
12	0.001	0.508	0.062	0.101	0.011	0.558	#21	0.054	Not Required	Pass
13	0.008	0.073	0.301	0.057	0.016	0.308	#24	0.306	Not Required	Pass
14	0.003	0.081	0.294	0.052	0.016	0.315	#24	0.204	Not Required	Pass
15	0.000	0.005	0.007	0.008	0.002	0.012	#21	Not Required	Not Required	Pass
16	0.000	0.005	0.007	0.008	0.002	0.012	#21	Not Required	Not Required	Pass
101	0.155	0.421	0.008	0.017	0.001	0.461	#13	0.619	Not Required	Pass
102	0.002	0.601	0.081	0.126	0.012	0.675	#21	0.054	Not Required	Pass
103	0.006	0.866	0.042	0.086	0.006	0.911	#21	0.046	Not Required	Pass
104	0.007	0.843	0.119	0.084	0.025	0.921	#21	0.082	Not Required	Pass
105	0.007	0.537	0.125	0.086	0.032	0.569	#21	0.076	Not Required	Pass
106	0.006	0.855	0.037	0.085	0.006	0.888	#21	0.046	Not Required	Pass
107	0.006	0.531	0.107	0.085	0.028	0.560	#21	0.076	Not Required	Pass
108	0.003	0.056	0.109	0.047	0.013	0.165	#21	0.102	Not Required	Pass
109	0.012	0.083	0.035	0.001	0.001	0.124	#21	0.206	Not Required	Pass
110	0.006	0.816	0.105	0.082	0.023	0.890	#21	0.082	Not Required	Pass

111	0.005	0.059	0.112	0.049	0.013	0.155	#21	0.102	Not Required	Pass
112	0.002	0.581	0.082	0.123	0.014	0.654	#21	0.054	Not Required	Pass
113	0.008	0.254	0.338	0.068	0.017	0.548	#21	0.306	Not Required	Pass
114	0.007	0.278	0.337	0.068	0.017	0.570	#21	0.306	Not Required	Pass
115	0.010	0.445	0.158	0.055	0.013	0.611	#21	0.507	Not Required	Pass
116	0.004	0.416	0.160	0.054	0.014	0.571	#21	0.507	Not Required	Pass
201	0.155	0.421	0.008	0.017	0.001	0.461	#13	0.619	Not Required	Pass
202	0.002	0.581	0.082	0.123	0.014	0.654	#21	0.054	Not Required	Pass
203	0.006	0.855	0.037	0.085	0.006	0.888	#21	0.046	Not Required	Pass
204	0.006	0.816	0.105	0.082	0.023	0.890	#21	0.082	Not Required	Pass
205	0.006	0.531	0.107	0.085	0.028	0.560	#21	0.076	Not Required	Pass
206	0.006	0.866	0.042	0.086	0.006	0.911	#21	0.046	Not Required	Pass
207	0.007	0.537	0.125	0.086	0.032	0.569	#21	0.076	Not Required	Pass
208	0.002	0.082	0.150	0.054	0.014	0.183	#21	0.102	Not Required	Pass
209	0.012	0.083	0.035	0.001	0.001	0.124	#21	0.206	Not Required	Pass
210	0.007	0.843	0.119	0.084	0.025	0.921	#21	0.082	Not Required	Pass
211	0.005	0.100	0.152	0.055	0.013	0.169	#21	0.102	Not Required	Pass
212	0.002	0.601	0.081	0.126	0.012	0.675	#21	0.054	Not Required	Pass
213	0.008	0.254	0.338	0.068	0.017	0.548	#21	0.306	Not Required	Pass
214	0.007	0.278	0.337	0.068	0.017	0.569	#21	0.306	Not Required	Pass
215	0.010	0.287	0.159	0.049	0.013	0.451	#21	0.507	Not Required	Pass
216	0.005	0.236	0.158	0.047	0.013	0.393	#21	0.507	Not Required	Pass
301	0.106	0.348	0.077	0.012	0.006	0.356	#16	0.619	Not Required	Pass
302	0.001	0.508	0.062	0.101	0.011	0.558	#21	0.054	Not Required	Pass
303	0.006	0.698	0.074	0.071	0.021	0.775	#21	0.046	Not Required	Pass
304	0.007	0.645	0.095	0.065	0.020	0.679	#21	0.082	Not Required	Pass
305	0.007	0.432	0.113	0.069	0.029	0.459	#21	0.076	Not Required	Pass
306	0.004	0.469	0.012	0.045	0.006	0.475	#21	0.046	Not Required	Pass
307	0.003	0.291	0.012	0.047	0.003	0.298	#21	0.076	Not Required	Pass
308	0.000	0.005	0.007	0.008	0.002	0.012	#21	Not Required	Not Required	Pass
309	0.004	0.084	0.060	0.004	0.004	0.145	#21	0.206	Not Required	Pass
310	0.002	0.445	0.046	0.045	0.010	0.492	#21	0.122	Not Required	Pass
311	0.000	0.005	0.007	0.008	0.002	0.012	#21	Not Required	Not Required	Pass
312	0.002	0.273	0.044	0.066	0.008	0.308	#21	0.054	Not Required	Pass
313	0.008	0.073	0.301	0.057	0.016	0.307	#24	0.204	Not Required	Pass
314	0.003	0.081	0.294	0.052	0.016	0.315	#24	0.306	Not Required	Pass
315	0.010	0.476	0.158	0.043	0.013	0.637	#21	0.507	Not Required	Pass
316	0.004	0.452	0.158	0.039	0.013	0.610	#21	0.507	Not Required	Pass

Definitions

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

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

REFERENCES	CALCULATIONS	RESULTS
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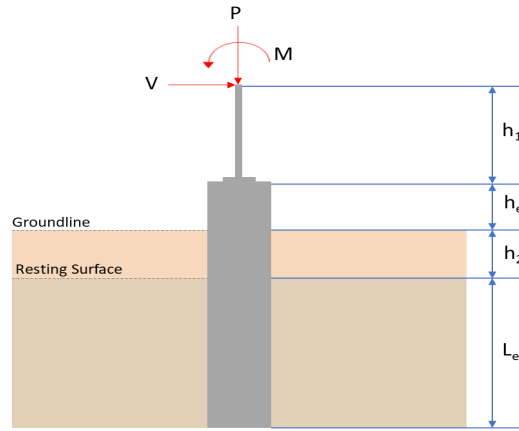
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.173	13.098
V_x (kip)	-0.845	-1.428
V_z (kip)	0.399	0.649
M_x (kipft)	1.814	2.964
M_z (kipft)	16.642	29.020

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.845 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13455 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.642 \text{ kipft}) + ((-0.845 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.65 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5124 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.399 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.063535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.814 \text{ kipft}) + ((0.399 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28885 \text{ kipft/ft}$$

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:

$L_{e,z} = 3.2914 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5124 \text{ ft}), (3.2914 \text{ ft})]$$

$$L_{e,req} = 5.512 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.512 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95861$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.173 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51081 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25541$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.13455 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.65 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.65 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.65 \text{ kipft/ft})) + (4 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9114 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.65 \text{ kipft/ft})) + (3 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.65 \text{ kipft/ft})) + (2 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24284 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.65 \text{ kipft/ft})) + ((-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.82141 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9114 \text{ ft})}{2}$$

$$p_a = 0.29336 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.24284 \text{ kip/ft}^2)}{(0.29336 \text{ kip/ft}^2)}$$

$$Ratio = 0.8278$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.82141 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.95236$$

Status: **PASS**
Ratio: **0.830**

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

Considering z-direction:

$H_o = 0.063535 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.28885 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28885 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.063535 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.28885 \text{ kipft/ft})) + (4 \times (0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0525 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.28885 \text{ kipft/ft})) + (3 \times (0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.28885 \text{ kipft/ft})) + (2 \times (0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.071989 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.28885 \text{ kipft/ft})) + ((0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.17114 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0525 \text{ ft})}{2}$$

$$p_a = 0.30394 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.071989 \text{ kip/ft}^2)}{(0.30394 \text{ kip/ft}^2)}$$

$$Ratio = 0.23685$$

p_s - Allowable lateral soil pressure at depth L_e .

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

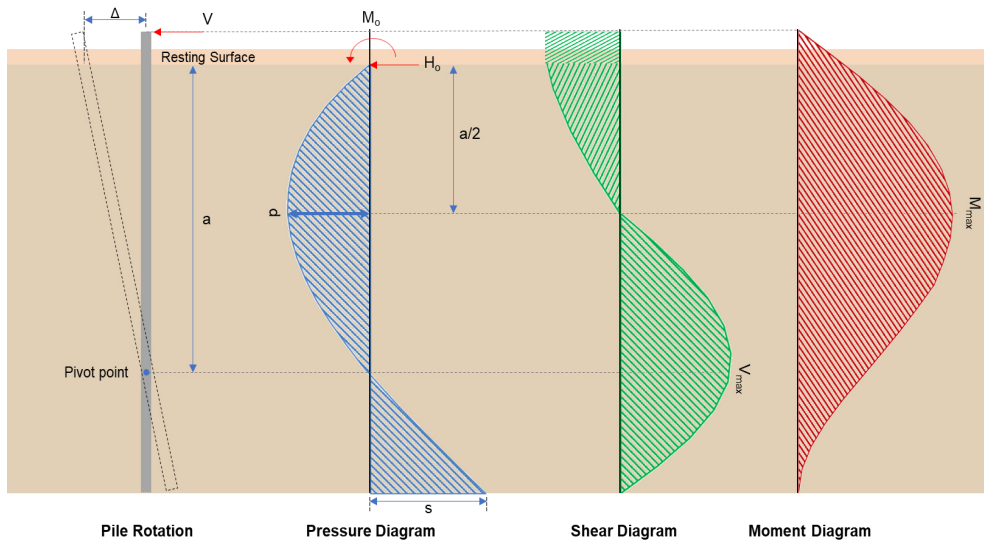
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.17114 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.19842$$

Status: **PASS**
Ratio: **0.240**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.428 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.22739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.02 \text{ kipft}) + ((-1.428 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.621 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.621 \text{ kipft/ft})}{(-0.22739 \text{ kip/ft})}$$

$$E = 20.322 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.621 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.621 \text{ kipft/ft})) + (4 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (4.621 \text{ kipft/ft})) + (4 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9094 \text{ ft}$$

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

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

$$V_{max} = ((-0.22739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9094 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9094 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.2956 \text{ kip}$$

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

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

$$M_{max} = ((-0.22739 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(20.322 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 17.621 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.649 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.10334 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.964 \text{ kipft}) + ((0.649 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.47197 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.47197 \text{ kipft/ft})}{(0.10334 \text{ kip/ft})}$$

$$E = 4.567 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.47197 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.10334 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.47197 \text{ kipft/ft})) + (4 \times (0.10334 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.052 \text{ ft}$$

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

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

$$V_{max} = ((0.10334 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.052 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.052 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.85465 \text{ kip}$$

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

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

$$M_{max} = ((0.10334 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.567 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.2437 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.098 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.161 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.161 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(13.098 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0048961$</p>	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p>	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.098 \text{ kip} \rightarrow 13098 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13098 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.23 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.23 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.23 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.23 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.23 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.2956 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.2956 \text{ kip})}{(111.23 \text{ kip})}$$

$$Ratio = 0.056599$$

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

Considering z-direction:

$V_{max} = 0.85465 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.85465 \text{ kip})}{(111.23 \text{ kip})}$$

$$Ratio = 0.0076835$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 17.621 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(17.621 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.070598$$

Status: **PASS**
Ratio: **0.070**

Considering z-direction:

$M_{max} = 2.2437 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.2437 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0089892$$

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

REFERENCES	CALCULATIONS	RESULTS
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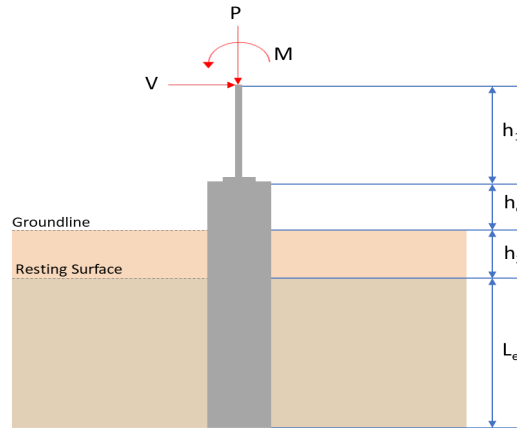
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 5.75$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	8.173	13.098
V_x (kip)	-0.845	-1.428
V_z (kip)	-0.399	-0.649
M_x (kipft)	-1.815	-2.964
M_z (kipft)	16.642	29.020

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.845 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13455 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(16.642 \text{ kipft}) + ((-0.845 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.65 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.5124 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.399 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.063535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.815 \text{ kipft}) + ((-0.399 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.28901 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.407 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.5124 \text{ ft}), (2.407 \text{ ft})]$$

$$L_{e,req} = 5.512 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5.75 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.512 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.95861$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.173 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51081 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51081 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25541$$

Status: **PASS**
Ratio: **0.260**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.4375$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.13455 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.65 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.65 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.65 \text{ kipft/ft})) + (4 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9114 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.65 \text{ kipft/ft})) + (3 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.65 \text{ kipft/ft})) + (2 \times (-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.24284 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.65 \text{ kipft/ft})) + ((-0.13455 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.82141 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.9114 \text{ ft})}{2}$$

$$p_a = 0.29336 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24284 \text{ kip/ft}^2)}{(0.29336 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.8278$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.82141 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95236$$

Status: **PASS**
Ratio: **0.830**

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

Considering z-direction:

$H_o = -0.063535 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.28901 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.28901 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.28901 \text{ kipft/ft})) + (4 \times (-0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.0525 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.28901 \text{ kipft/ft})) + (3 \times (-0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (0.28901 \text{ kipft/ft})) + (2 \times (-0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.0006002 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.28901 \text{ kipft/ft})) + ((-0.063535 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.038599 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0525 \text{ ft})}{2}$$

$$p_a = 0.30393 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0006002 \text{ kip/ft}^2)}{(0.30393 \text{ kip/ft}^2)}$$

$$Ratio = 0.0019748$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$$

$$p_s = 0.8625 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

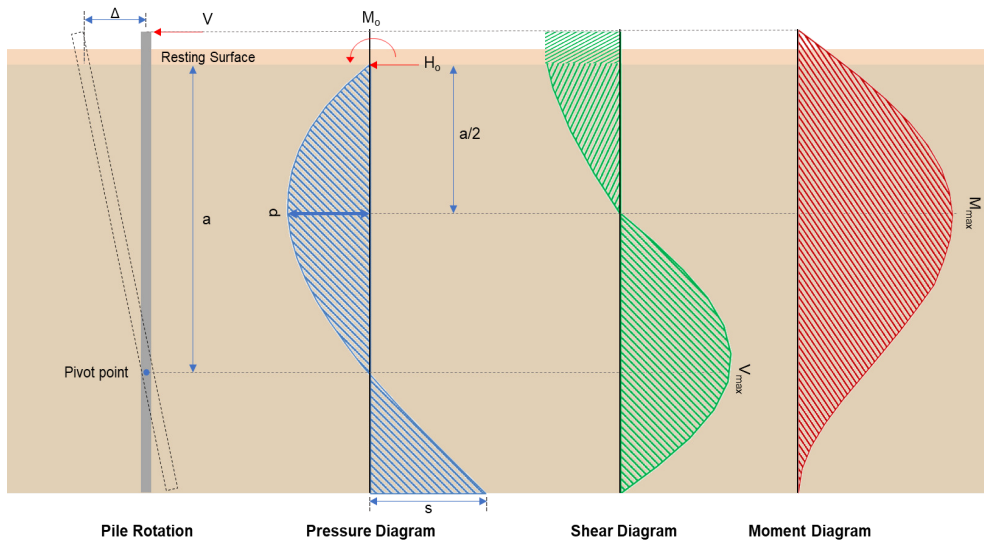
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.038599 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = 0.044753$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.428 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.22739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(29.02 \text{ kipft}) + ((-1.428 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.621 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.621 \text{ kipft/ft})}{(-0.22739 \text{ kip/ft})}$$

$$E = 20.322 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.621 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.621 \text{ kipft/ft})) + (4 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = \frac{(-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft})}{(6 \times (4.621 \text{ kipft/ft})) + (4 \times (-0.22739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.9094 \text{ ft}$$

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

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

$$V_{max} = ((-0.22739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9094 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9094 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.2956 \text{ kip}$$

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

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

$$M_{max} = ((-0.22739 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(20.322 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (20.322 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.9094 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 17.621 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.649 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.10334 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(2.964 \text{ kipft}) + ((-0.649 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.47197 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.47197 \text{ kipft/ft})}{(-0.10334 \text{ kip/ft})}$$

$$E = 4.567 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.47197 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.10334 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.47197 \text{ kipft/ft})) + (4 \times (-0.10334 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.052 \text{ ft}$$

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

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

$$V_{max} = ((-0.10334 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.052 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.052 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.85465 \text{ kip}$$

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

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

$$M_{max} = ((-0.10334 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(4.567 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.567 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.052 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 2.2437 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.098 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.161 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.161 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(13.098 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0048961$	<p>Status: PASS Ratio: 0.000</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.098 \text{ kip} \rightarrow 13098 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(13098 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.23 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.23 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.23 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.23 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.23 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.2956 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.2956 \text{ kip})}{(111.23 \text{ kip})}$$

$$Ratio = 0.056599$$

Considering z-direction:

$V_{max} = 0.85465 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.85465 \text{ kip})}{(111.23 \text{ kip})}$$

$$Ratio = 0.0076835$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 17.621 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(17.621 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.070598$$

Status: **PASS**
 Ratio: **0.070**

Considering z-direction:

$M_{max} = 2.2437 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(2.2437 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0089892$$

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

REFERENCES	CALCULATIONS	RESULTS
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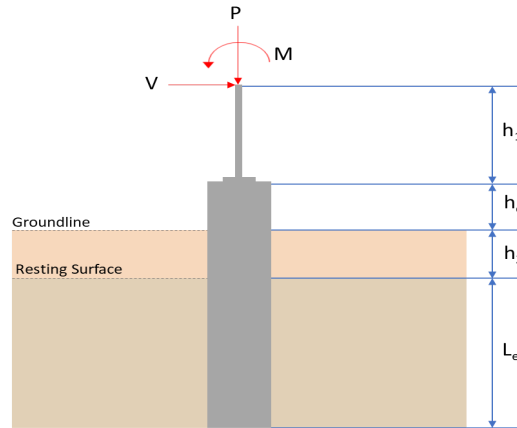
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.894	19.200
V_x (kip)	-1.169	-1.965
V_z (kip)	-0.037	-0.065
M_x (kipft)	-0.168	-0.296
M_z (kipft)	20.496	35.056

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.496 \text{ kipft}) + ((-1.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2637 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.8107 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0058917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.168 \text{ kipft}) + ((-0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026752 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1974 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.8107 \text{ ft}), (1.1974 \text{ ft})]$$

$$L_{e,req} = 5.811 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$Ratio = \frac{(5.811 \text{ ft})}{(6.25 \text{ ft})}$$

$$Ratio = 0.92976$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(11.894 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.74337 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.74337 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37169$$

Status: **PASS**
Ratio: **0.370**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18615 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.2637 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2637 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.2637 \text{ kipft/ft})) + (4 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2667 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2637 \text{ kipft/ft})) + (3 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.2637 \text{ kipft/ft})) + (2 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.23531 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.2637 \text{ kipft/ft})) + ((-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.82391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2667 \text{ ft})}{2}$$

$$p_a = 0.32 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23531 \text{ kip/ft}^2)}{(0.32 \text{ kip/ft}^2)}$$

$$Ratio = 0.73535$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.82391 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.87883$$

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

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = -0.0058917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.026752 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.026752 \text{ kipft/ft})) + (4 \times (-0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4159 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.026752 \text{ kipft/ft})) + (3 \times (-0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.026752 \text{ kipft/ft})) + (2 \times (-0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.000034851 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.026752 \text{ kipft/ft})) + ((-0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.002562 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4159 \text{ ft})}{2}$$

$$p_a = 0.33119 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.000034851 \text{ kip/ft}^2)}{(0.33119 \text{ kip/ft}^2)}$$

$$Ratio = 0.00010523$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

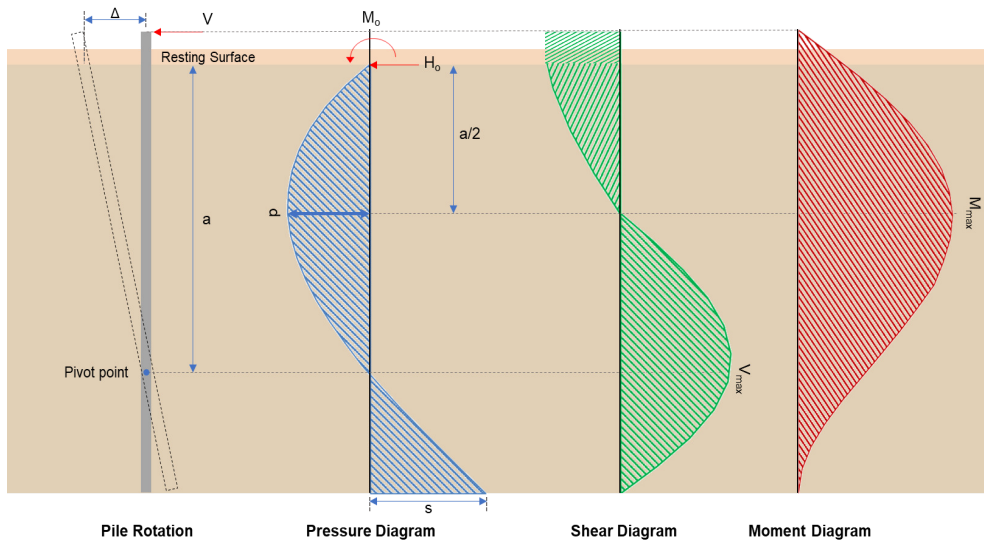
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.002562 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.0027328$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.965 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.3129 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.056 \text{ kipft}) + ((-1.965 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5822 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5822 \text{ kipft/ft})}{(-0.3129 \text{ kip/ft})}$$

$$E = 17.84 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5822 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.5822 \text{ kipft/ft})) + (4 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{6 \times (5.5822 \text{ kipft/ft}) + (4 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft}))}{\dots}$$

$$a = 4.2653 \text{ ft}$$

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

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

$$V_{max} = ((-0.3129 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.1526 \text{ kip}$$

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

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

$$M_{max} = ((-0.3129 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(17.84 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.637 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.065 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.01035 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.296 \text{ kipft}) + ((-0.065 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.047134 \text{ kipft/ft})}{(-0.01035 \text{ kip/ft})}$$

$$E = 4.5538 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.047134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.01035 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.047134 \text{ kipft/ft})) + (4 \times (-0.01035 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4155 \text{ ft}$$

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

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

$$V_{max} = ((-0.01035 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.080816 \text{ kip}$$

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

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

$$M_{max} = ((-0.01035 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.5538 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.22935 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(19.2 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.958 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.958 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(19.2 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0071771$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 19.2 \text{ kip} \rightarrow 19200 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(19200 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.05 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.05 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.05 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((121.05 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.76 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.1526 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.1526 \text{ kip})}{(111.76 \text{ kip})}$$

$$Ratio = 0.064$$

Considering z-direction:

$V_{max} = 0.080816 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.080816 \text{ kip})}{(111.76 \text{ kip})}$$

$$Ratio = 0.00072312$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 21.637 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(21.637 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.086688$$

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

Considering z-direction:

$M_{max} = 0.22935 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.22935 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00091886$$

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

REFERENCES	CALCULATIONS	RESULTS
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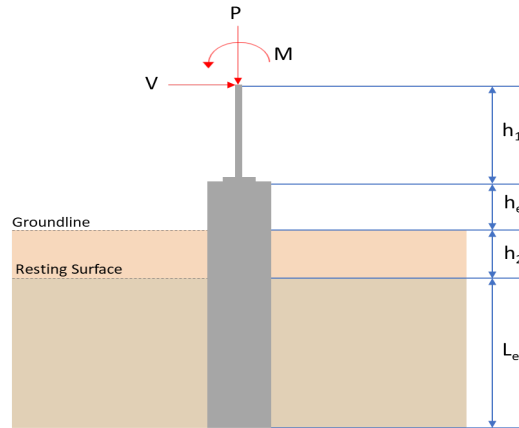
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

Load Component	ASD	LRFD
P (kip)	11.894	19.200
V_x (kip)	-1.169	-1.965
V_z (kip)	0.037	0.065
M_x (kipft)	0.168	0.296
M_z (kipft)	20.496	35.056

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.18615 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.496 \text{ kipft}) + ((-1.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2637 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.8107 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.037 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0058917 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.168 \text{ kipft}) + ((0.037 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.026752 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3799 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.8107 \text{ ft}), (1.3799 \text{ ft})]$$

$$L_{e,req} = 5.811 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.811 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.92976$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(11.894 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.74337 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.74337 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.37169$$

Status: **PASS**
Ratio: **0.370**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.18615 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.2637 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2637 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.2637 \text{ kipft/ft})) + (4 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2667 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2637 \text{ kipft/ft})) + (3 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.2637 \text{ kipft/ft})) + (2 \times (-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.23531 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.2637 \text{ kipft/ft})) + ((-0.18615 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.82391 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2667 \text{ ft})}{2}$$

$$p_a = 0.32 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23531 \text{ kip/ft}^2)}{(0.32 \text{ kip/ft}^2)}$$

$$Ratio = 0.73535$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.82391 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.87883$$

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

Status: **PASS**
Ratio: **0.880**

Considering z-direction:

$H_o = 0.0058917 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.026752 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.026752 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.026752 \text{ kipft/ft})) + (4 \times (0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4159 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.026752 \text{ kipft/ft})) + (3 \times (0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.026752 \text{ kipft/ft})) + (2 \times (0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0059004 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.026752 \text{ kipft/ft})) + ((0.0058917 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.013874 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4159 \text{ ft})}{2}$$

$$p_a = 0.33119 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0059004 \text{ kip/ft}^2)}{(0.33119 \text{ kip/ft}^2)}$$

$$Ratio = 0.017816$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

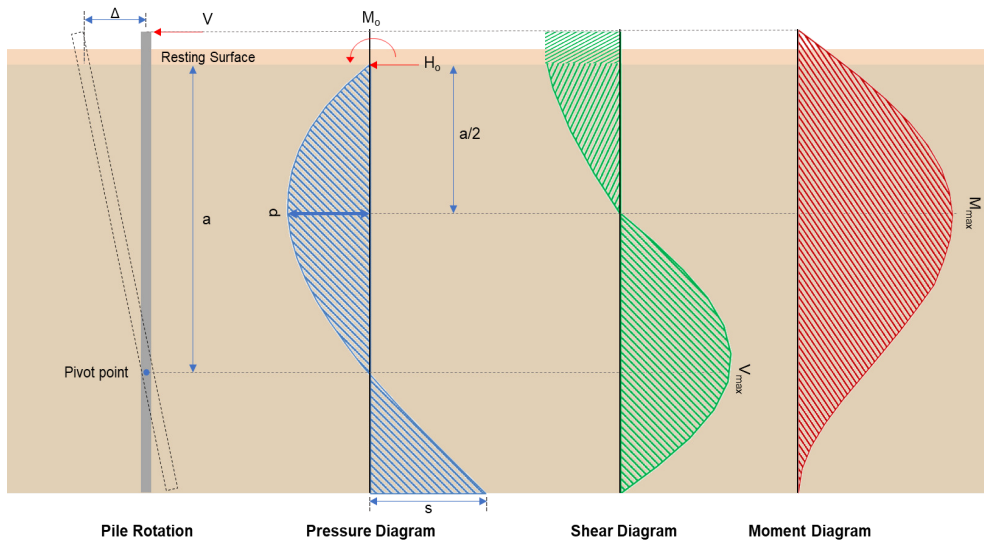
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.013874 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.014799$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.965 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.3129 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.056 \text{ kipft}) + ((-1.965 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.5822 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5822 \text{ kipft/ft})}{(-0.3129 \text{ kip/ft})}$$

$$E = 17.84 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5822 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.5822 \text{ kipft/ft})) + (4 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(6 \times (5.5822 \text{ kipft/ft})) + (4 \times (-0.3129 \text{ kip/ft}) \times (6.25 \text{ ft}))}{}$$

$$a = 4.2653 \text{ ft}$$

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

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

$$V_{max} = ((-0.3129 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.1526 \text{ kip}$$

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

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

$$M_{max} = ((-0.3129 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(17.84 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (17.84 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2653 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.637 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.065 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.01035 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.296 \text{ kipft}) + ((0.065 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

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

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.047134 \text{ kipft/ft})}{(0.01035 \text{ kip/ft})}$$

$$E = 4.5538 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.047134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.01035 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.047134 \text{ kipft/ft})) + (4 \times (0.01035 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4155 \text{ ft}$$

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

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

$$V_{max} = ((0.01035 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.080816 \text{ kip}$$

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

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

$$M_{max} = ((0.01035 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.5538 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.5538 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4155 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.22935 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(19.2 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -83.958 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-83.958 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

<p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p>	<p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p>	<p>Status: PASS Ratio: 0.970</p>
<p>22.4.2.2</p>	<p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(19.2 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0071771$	<p>Status: PASS Ratio: 0.010</p>
<p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p>	<p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$	

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 19.2 \text{ kip} \rightarrow 19200 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(19200 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 121.05 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (121.05 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 121.05 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((121.05 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.76 \text{ kip}$$

Considering x-direction:

$V_{max} = 7.1526 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(7.1526 \text{ kip})}{(111.76 \text{ kip})}$$

$$Ratio = 0.064$$

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

Considering z-direction:

$V_{max} = 0.080816 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.080816 \text{ kip})}{(111.76 \text{ kip})}$$

$$Ratio = 0.00072312$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 21.637 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(21.637 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.086688$$

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

Considering z-direction:

$M_{max} = 0.22935 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.22935 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00091886$$

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