

Your Project Calculations



Project Name: MTSOLAR_H9539J521882

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_H9539J521882&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/5_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=cyVGI3NwIAQmgmjbXmQUBVYLJ6ShBAS3oj8waUPCUXOp78e11PwR0Zzk6EIUUBwV

Array Specification

Product:	Beam
Unique ID:	5P-19.75-10TOP-XD-24-L-5Hx12W-E98F
Duty Classification:	XD
Module Width:	44.65 in
Module Length:	89.69in
Number of Rows:	5
Number of Columns:	12
Total Number of Modules:	60
Desired Tilt Angle:	7
Front Edge Clearance:	10
Total Array Height at Tilt:	12.28 ft
Total Frame Length:	90.50 ft
Frame Weight:	5792 lbs
Array Dimensions N/S:	18.81 ft
Array Dimensions E/W:	90.69 ft
Rail Length:	225.75 in
Rail Spacing:	3.74 ft
Rail Check:	Not Checked

Support Specifications

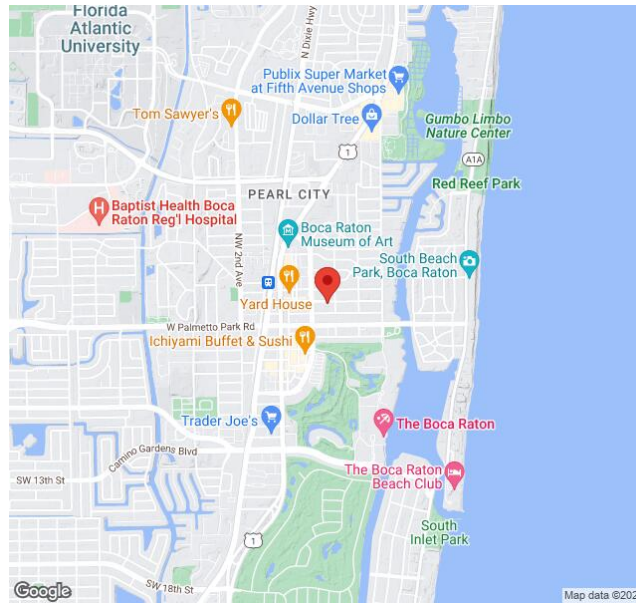
Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	11.15 ft
Number of Poles:	5
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Square
Foundation Dimensions:	48 x 48 in
Foundation Depth (below grade):	Pile 1: 5.75 ft Pile 2: 6.25 ft Pile 3: 6.25 ft Pile 4: 6.25 ft Pile 5: 5.75 ft
Foundation Volume:	17.926 y ³
Foundation Result:	PASSED
Mount Twist:	0.357229 kip

Site Info

Risk Category:	I
Exposure:	B
Soil Classification:	sand
Site Location:	266 NE 2nd St, Boca Raton, FL 33432, USA
Wind Speed:	160 mph
Snow Load:	0 psf
Design Uplift Pressure:	Multiple pressures
Design Downforce Pressure:	Multiple pressures
Design Snow Pressure:	0.000000 ksf



Design Disclaimer

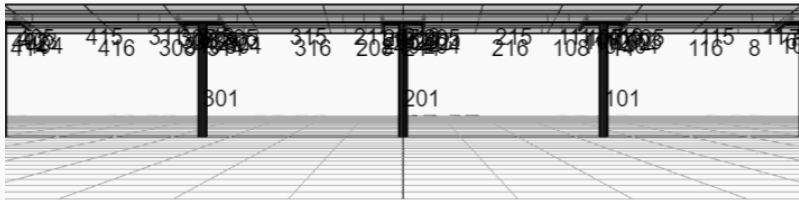
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

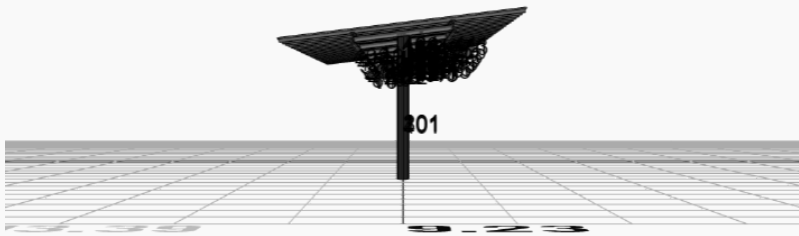
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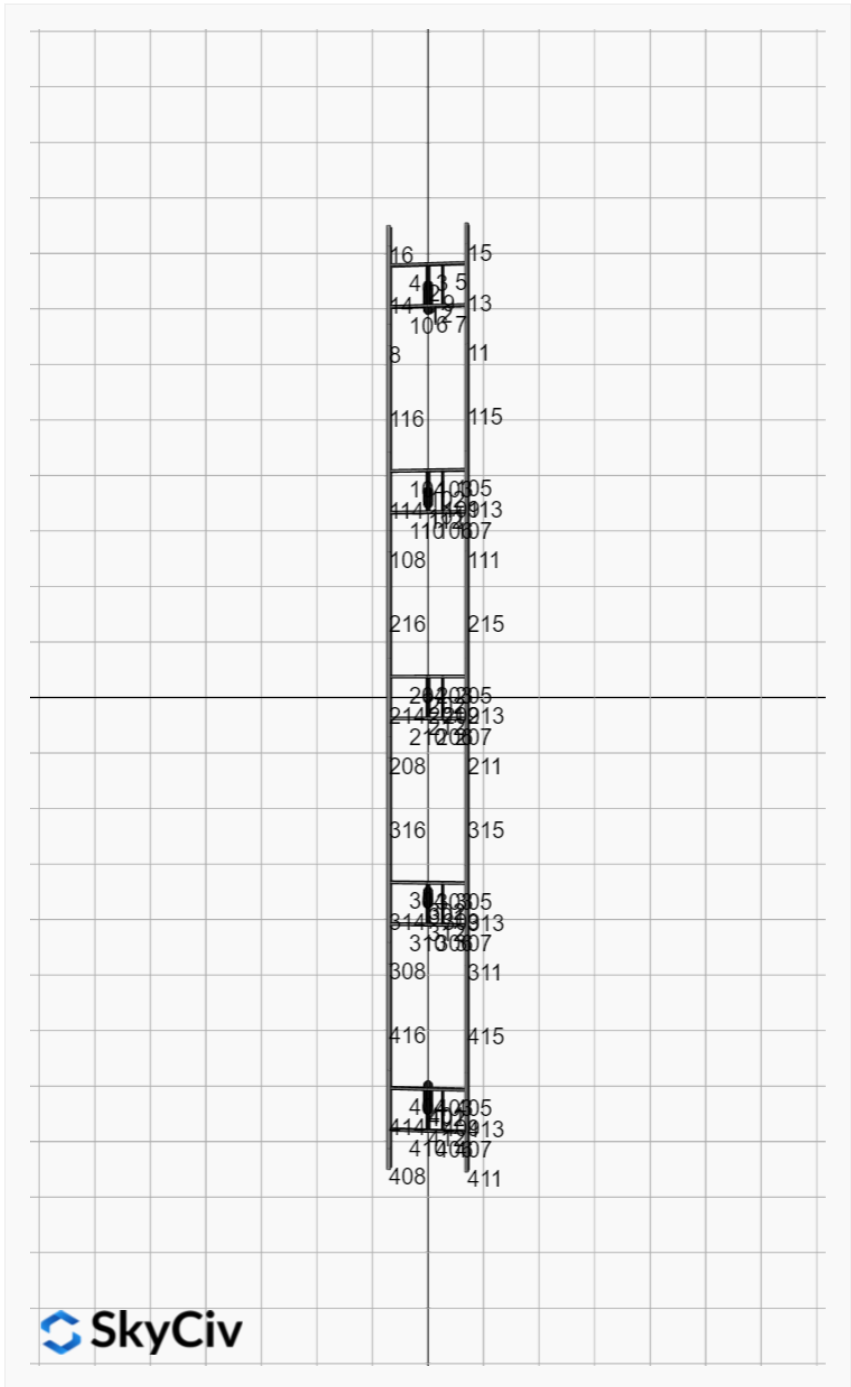
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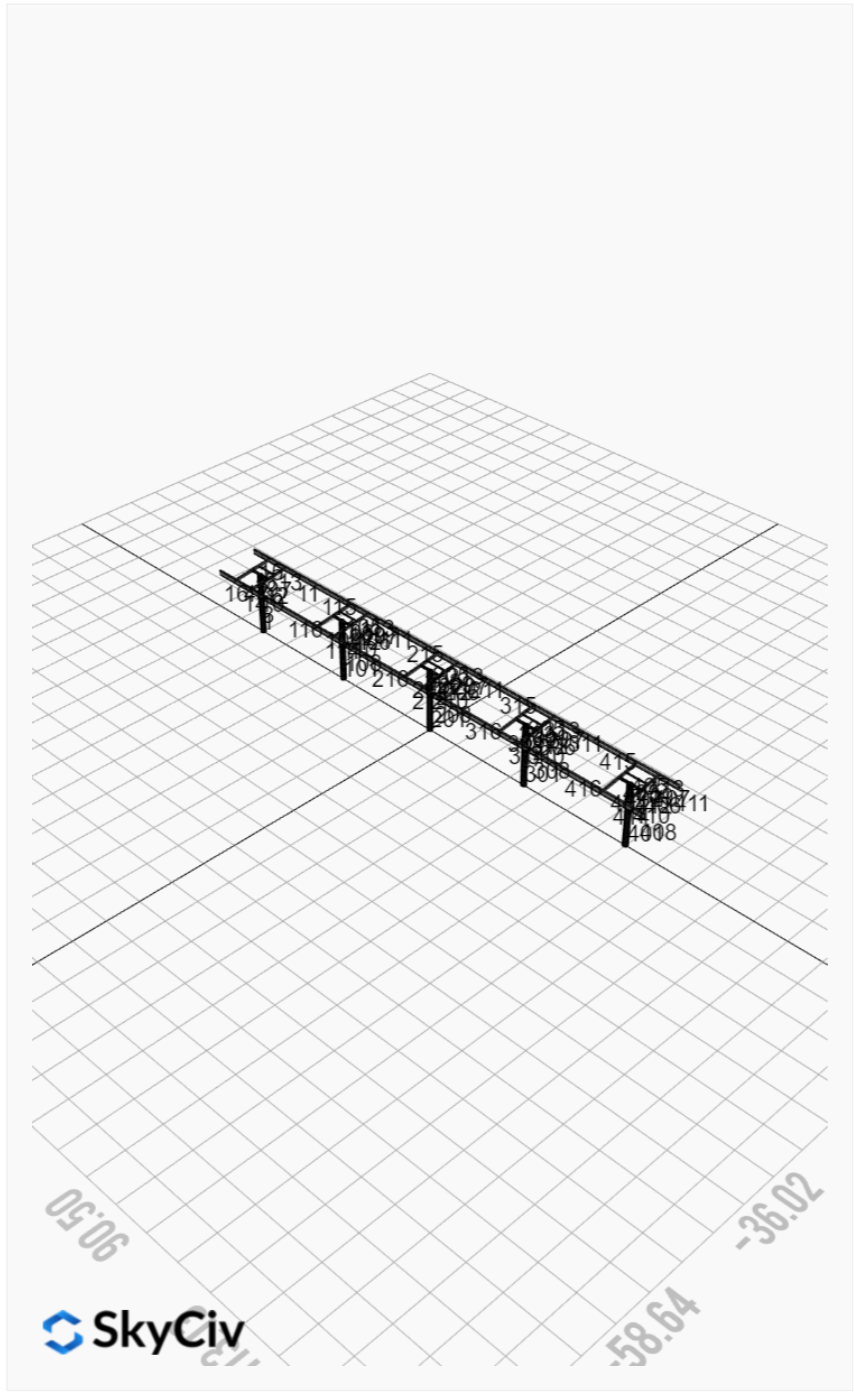
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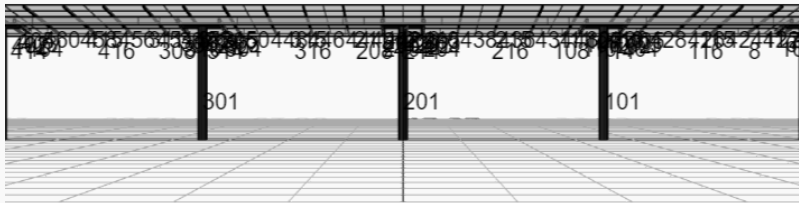
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only



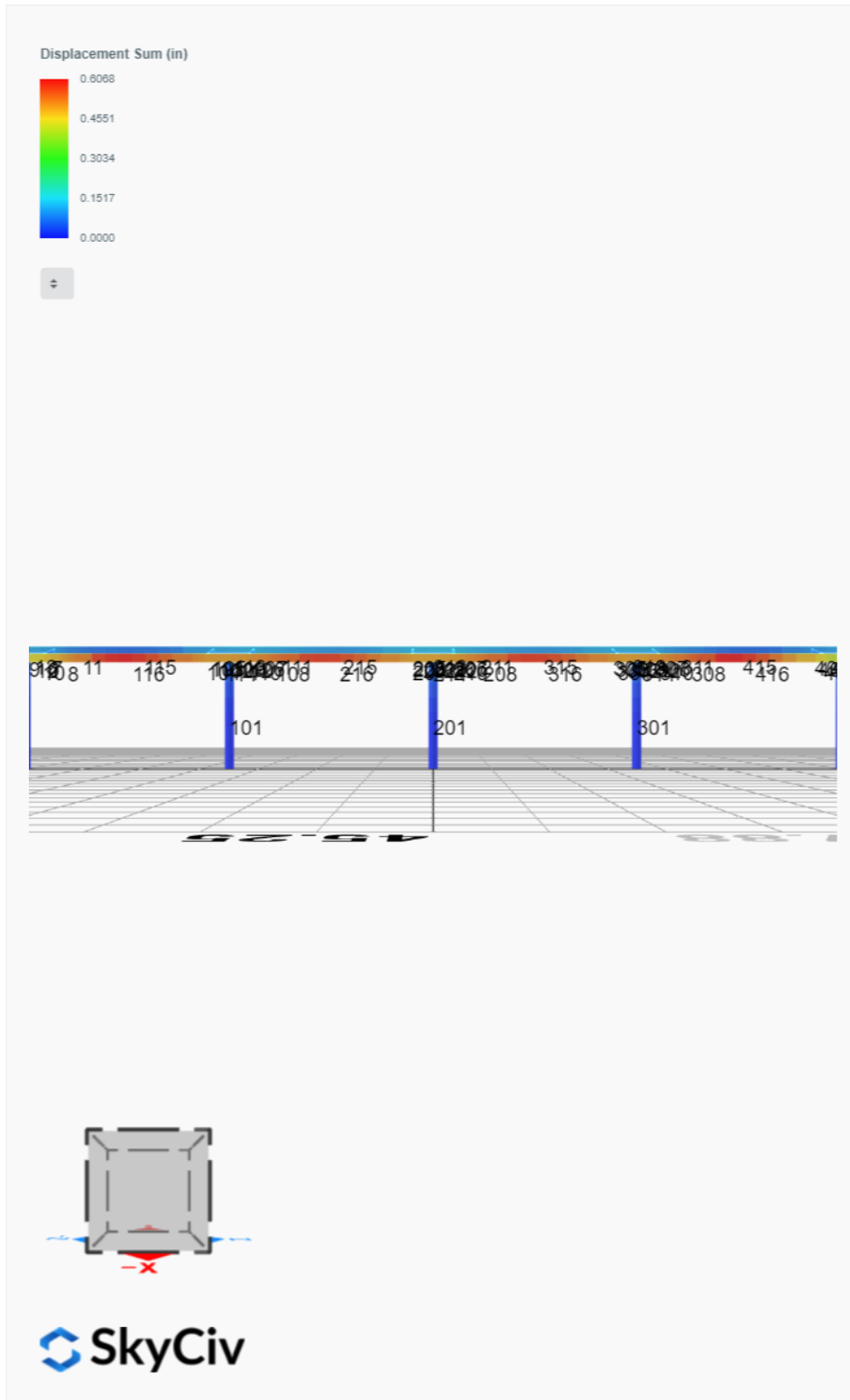




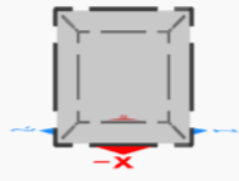
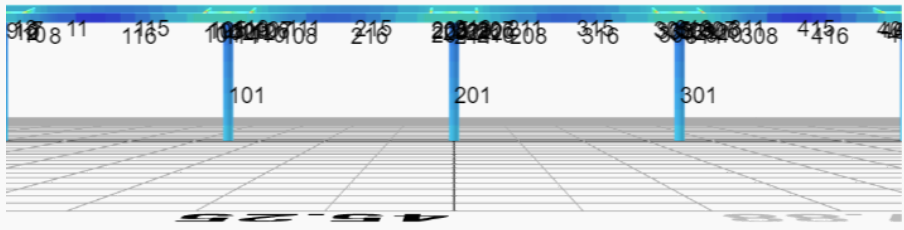




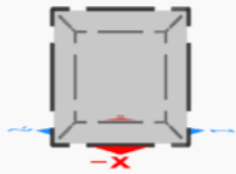
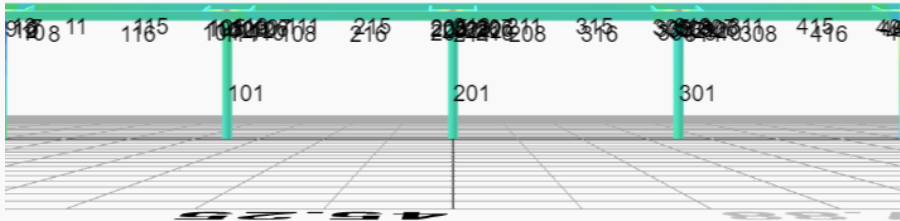
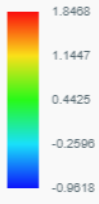
FEM Results (Envelope Worst Case for each member)



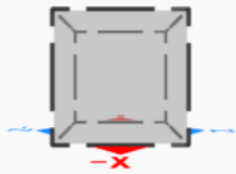
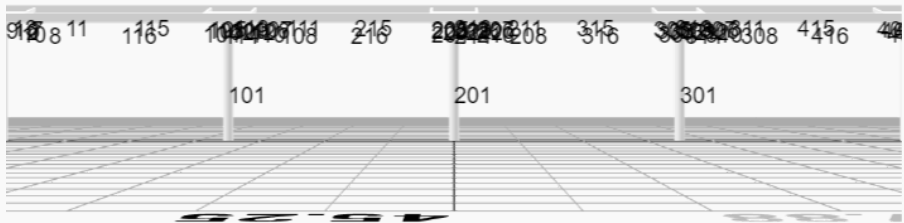
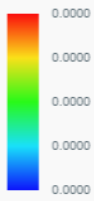
Top Bending Stress Z (ksi)



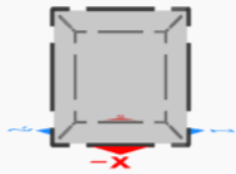
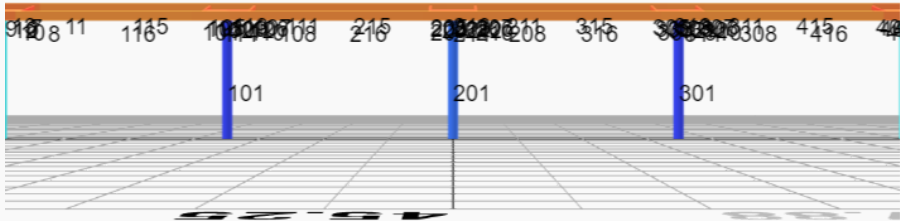
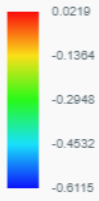
Top Bending Stress Y (ksi)



Shear Stress Y (ksi)



Axial Stress (ksi)



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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 2. D + L	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 3. D + (S or Lr or R)	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 3. D + (S or Lr or R)	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 5b. D + 0.7E	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0085	2.3280	0.0753	0.2016	-0.0135	-0.0572
ULS: 8. 0.6D + 0.7E	0.0051	1.3968	0.0452	0.1210	-0.0081	-0.0343
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6599	7.6666	0.3314	0.8726	-0.2179	12.0328
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6599	7.6666	0.3314	0.8726	-0.2179	12.0328
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3976	-0.8472	-0.0717	-0.1842	0.0938	-0.5949
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4168	-0.8408	-0.0823	-0.2094	0.1267	-16.6433
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4928	6.3319	0.2674	0.7049	-0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4928	6.3319	0.2674	0.7049	-0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3003	-0.0534	-0.0350	-0.0877	0.0669	-0.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3147	-0.0486	-0.0429	-0.1066	0.0917	-12.4968
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4928	6.3319	0.2674	0.7049	-0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4928	6.3319	0.2674	0.7049	-0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3003	-0.0534	-0.0350	-0.0877	0.0669	-0.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3147	-0.0486	-0.0429	-0.1066	0.0917	-12.4968
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6632	6.7354	0.3013	0.7920	-0.2125	12.0557
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6632	6.7354	0.3013	0.7920	-0.2125	12.0557
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3943	-1.7784	-0.1018	-0.2648	0.0992	-0.5720
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4134	-1.7720	-0.1124	-0.2901	0.1322	-16.6204

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	11.6912
Shear X	-1.1139
Shear Z	0.5179
Moment X	1.3623
Moment Y (Twist)	0.3571
Moment Z	27.8522

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	7.6666
Shear X	-0.6632
Shear Z	0.3314
Moment X	0.8726
Moment Y (Twist)	0.2179
Moment Z	16.6433

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	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 2. D + L	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 3. D + (S or Lr or R)	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 3. D + (S or Lr or R)	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 5b. D + 0.7E	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0093	2.9268	-0.0097	-0.0292	0.0075	0.1277
ULS: 8. 0.6D + 0.7E	-0.0056	1.7561	-0.0058	-0.0175	0.0045	0.0766
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8961	10.2552	-0.0402	-0.1228	0.0127	15.9632
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8961	10.2552	-0.0402	-0.1228	0.0127	15.9632
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5272	-1.4426	0.0104	0.0304	-0.0077	-0.8589
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5046	-1.4081	0.0056	0.0205	0.0211	-21.0339
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6744	8.4231	-0.0325	-0.0994	0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6744	8.4231	-0.0325	-0.0994	0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3931	-0.3503	0.0053	0.0155	-0.0039	-0.6123
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3761	-0.3244	0.0018	0.0081	0.0177	-15.7435
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6744	8.4231	-0.0325	-0.0994	0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6744	8.4231	-0.0325	-0.0994	0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3931	-0.3503	0.0053	0.0155	-0.0039	-0.6123
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3761	-0.3244	0.0018	0.0081	0.0177	-15.7435
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8924	9.0845	-0.0363	-0.1111	0.0097	15.9121
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8924	9.0845	-0.0363	-0.1111	0.0097	15.9121
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5309	-2.6133	0.0142	0.0421	-0.0107	-0.9100
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5083	-2.5788	0.0095	0.0322	0.0181	-21.0849

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	15.7263
Shear X	-1.4891
Shear Z	-0.0625
Moment X	-0.1913
Moment Y (Twist)	0.0320
Moment Z	35.3319

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	10.2552
Shear X	-0.8961
Shear Z	-0.0402
Moment X	-0.1228
Moment Y (Twist)	0.0211
Moment Z	21.0849

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	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 2. D + L	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 3. D + (S or Lr or R)	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 3. D + (S or Lr or R)	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 5b. D + 0.7E	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0017	2.8268	-0.0000	0.0000	0.0000	0.0225
ULS: 8. 0.6D + 0.7E	0.0010	1.6961	-0.0000	0.0000	0.0000	0.0135
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8583	9.8286	-0.0000	-0.0000	0.0000	15.7221
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8583	9.8286	-0.0000	-0.0000	0.0000	15.7221
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5145	-1.3383	-0.0000	0.0000	0.0000	-0.7179
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5101	-1.3281	-0.0000	0.0000	0.0000	-21.3096
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6433	8.0782	-0.0000	-0.0000	0.0000	11.7972
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6433	8.0782	-0.0000	-0.0000	0.0000	11.7972
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3863	-0.2970	-0.0000	0.0000	0.0000	-0.5328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3830	-0.2893	-0.0000	0.0000	0.0000	-15.9766

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	-0.6433	8.0782	-0.0000	-0.0000	0.0000	11.7972
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6433	8.0782	-0.0000	-0.0000	0.0000	11.7972
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3863	-0.2970	-0.0000	0.0000	0.0000	-0.5328
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3830	-0.2893	-0.0000	0.0000	0.0000	-15.9766
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8590	8.6979	-0.0000	-0.0000	0.0000	15.7130
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8590	8.6979	-0.0000	-0.0000	0.0000	15.7130
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5138	-2.4690	-0.0000	0.0000	0.0000	-0.7269
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5094	-2.4588	-0.0000	0.0000	0.0000	-21.3186

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	15.0620
Shear X	-1.4333
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	35.7428

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	9.8286
Shear X	-0.8590
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	21.3186

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	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 2. D + L	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 3. D + (S or Lr or R)	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 3. D + (S or Lr or R)	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 5b. D + 0.7E	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0093	2.9268	0.0097	0.0292	-0.0075	0.1277
ULS: 8. 0.6D + 0.7E	-0.0056	1.7561	0.0058	0.0175	-0.0045	0.0766
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.8961	10.2552	0.0402	0.1228	-0.0127	15.9632
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.8961	10.2552	0.0402	0.1228	-0.0127	15.9632
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.5272	-1.4426	-0.0104	-0.0304	0.0077	-0.8589
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.5046	-1.4081	-0.0056	-0.0205	-0.0211	-21.0339
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6744	8.4231	0.0325	0.0994	-0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6744	8.4231	0.0325	0.0994	-0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3931	-0.3503	-0.0053	-0.0155	0.0039	-0.6123
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3761	-0.3244	-0.0018	-0.0081	-0.0177	-15.7435
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.6744	8.4231	0.0325	0.0994	-0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.6744	8.4231	0.0325	0.0994	-0.0114	12.0043
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3931	-0.3503	-0.0053	-0.0155	0.0039	-0.6123
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3761	-0.3244	-0.0018	-0.0081	-0.0177	-15.7435
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.8924	9.0845	0.0363	0.1111	-0.0097	15.9121
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.8924	9.0845	0.0363	0.1111	-0.0097	15.9121
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.5309	-2.6133	-0.0142	-0.0421	0.0107	-0.9100
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.5083	-2.5788	-0.0095	-0.0322	-0.0181	-21.0849

Worst Case Reactions LRFD

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	15.7263
Shear X	-1.4891
Shear Z	0.0625
Moment X	0.1914
Moment Y (Twist)	0.0320
Moment Z	35.3319

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	10.2552
Shear X	-0.8961
Shear Z	0.0402
Moment X	0.1228
Moment Y (Twist)	0.0211
Moment Z	21.0849

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 2. D + L	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 3. D + (S or Lr or R)	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 3. D + (S or Lr or R)	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 5b. D + 0.7E	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0085	2.3280	-0.0753	-0.2016	0.0135	-0.0572
ULS: 8. 0.6D + 0.7E	0.0051	1.3968	-0.0452	-0.1210	0.0081	-0.0343
ULS: 5a. D + 0.6W_Wind downforce Case A only	-0.6599	7.6666	-0.3314	-0.8726	0.2179	12.0328
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.6599	7.6666	-0.3314	-0.8726	0.2179	12.0328
ULS: 5a. D + 0.6W_Wind uplift Case A only	0.3976	-0.8472	0.0717	0.1842	-0.0938	-0.5949
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.4168	-0.8408	0.0823	0.2094	-0.1267	-16.6433
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4928	6.3319	-0.2674	-0.7049	0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4928	6.3319	-0.2674	-0.7049	0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3003	-0.0534	0.0350	0.0877	-0.0669	-0.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3147	-0.0486	0.0429	0.1066	-0.0917	-12.4968
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.4928	6.3319	-0.2674	-0.7049	0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.4928	6.3319	-0.2674	-0.7049	0.1668	9.0103
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.3003	-0.0534	0.0350	0.0877	-0.0669	-0.4605
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.3147	-0.0486	0.0429	0.1066	-0.0917	-12.4968
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-0.6632	6.7354	-0.3013	-0.7920	0.2125	12.0557
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.6632	6.7354	-0.3013	-0.7920	0.2125	12.0557
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	0.3943	-1.7784	0.1018	0.2648	-0.0992	-0.5720
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.4134	-1.7720	0.1124	0.2901	-0.1321	-16.6204

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	11.6912
Shear X	-1.1139
Shear Z	-0.5179
Moment X	-1.3622
Moment Y (Twist)	0.3572
Moment Z	27.8524

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	7.6666
Shear X	-0.6632
Shear Z	-0.3314
Moment X	-0.8726
Moment Y (Twist)	0.2179
Moment Z	16.6433

Project Details

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

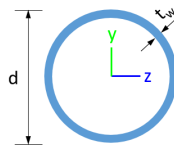


Design Input Information

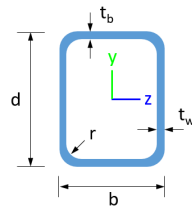
Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

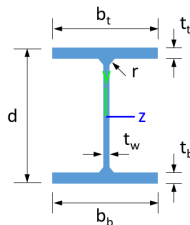
Section Dimensions



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				
11	10in Pipe Sch 40	10.75	0.36				



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)
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38

412	6	1.30	1.30	2.00	-	0	0	1
413	20	4.88	4.00	7.50	1.21,1.21,1.21,1.21,1.21,1.21,1.22,1.22,1.11,1.28,1.22,1.22,1.11,1.28,1.22,1.22,2.40,1.30,1.22,1.22,2.40,1.30,1.23,1.23,1.13,1.28	300	200	1
414	20	4.88	4.00	7.50	1.20,1.20,1.20,1.20,1.20,1.20,1.17,1.17,1.26,1.91,1.17,1.17,1.26,1.91,1.18,1.18,1.30,1.42,1.18,1.18,1.30,1.42,1.17,1.17,1.25,2.20	300	200	1
415	20	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.08,1.09,1.09,1.36,1.09,1.09,1.09,1.36,1.09,1.09,1.08	300	200	1
416	20	6.63	6.63	10.20	1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.12,1.09,1.09,1.09,1.12,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.10,1.09,1.09,1.14	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	535.87	349.50	147.68	147.68	160.76	160.76
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	142.47	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	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	33.92	6.46	56.26	44.91
14	159.30	116.35	35.76	6.46	56.26	44.91
15	159.30	113.66	46.90	6.46	56.26	44.91
16	159.30	113.66	46.90	6.46	56.26	44.91
101	535.87	349.50	147.68	147.68	160.76	160.76
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	142.47	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	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.48	6.46	56.26	44.91
114	159.30	116.35	31.78	6.46	56.26	44.91
115	159.30	75.13	21.57	6.46	56.26	44.91
116	159.30	75.13	20.61	6.46	56.26	44.91
201	535.87	349.50	147.68	147.68	160.76	160.76
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	142.47	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	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.48	6.46	56.26	44.91
214	159.30	116.35	31.78	6.46	56.26	44.91
215	159.30	75.13	21.96	6.46	56.26	44.91
216	159.30	75.13	22.15	6.46	56.26	44.91
301	535.87	349.50	147.68	147.68	160.76	160.76
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	142.47	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	142.47	46.90	6.46	56.26	44.91
312	251.01	248.88	27.16	27.16	75.30	75.30
313	159.30	116.35	31.48	6.46	56.26	44.91
314	159.30	116.35	31.78	6.46	56.26	44.91
315	159.30	75.13	21.38	6.46	56.26	44.91
316	159.30	75.13	21.18	6.46	56.26	44.91
401	535.87	349.50	147.68	147.68	160.76	160.76
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95
407	151.65	149.10	20.17	14.14	54.12	28.95
408	159.30	113.66	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	113.66	46.90	6.46	56.26	44.91
412	251.01	248.88	27.16	27.16	75.30	75.30
413	159.30	116.35	33.92	6.46	56.26	44.91
414	159.30	116.35	35.76	6.46	56.26	44.91
415	159.30	75.13	20.80	6.46	56.26	44.91
416	159.30	75.13	20.99	6.46	56.26	44.91

Design Ratio

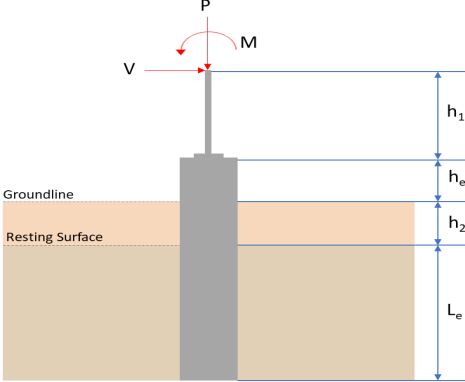
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.033	0.189	0.030	0.007	0.003	0.195	#32	0.382	Not Required	Pass
2	0.001	0.272	0.031	0.062	0.006	0.303	#13	0.054	Not Required	Pass
3	0.001	0.520	0.014	0.051	0.003	0.534	#13	0.046	Not Required	Pass
4	0.000	0.352	0.025	0.035	0.005	0.377	#13	0.122	Not Required	Pass
5	0.001	0.322	0.009	0.052	0.003	0.331	#13	0.076	Not Required	Pass
6	0.001	0.688	0.022	0.070	0.005	0.710	#13	0.046	Not Required	Pass
7	0.001	0.427	0.015	0.068	0.005	0.437	#13	0.076	Not Required	Pass
8	0.002	0.091	0.025	0.028	0.001	0.105	#13	0.102	Not Required	Pass

9	0.001	0.080	0.025	0.003	0.002	0.104	#13	0.206	Not Required	Pass
10	0.001	0.483	0.020	0.049	0.005	0.503	#13	0.082	Not Required	Pass
11	0.002	0.125	0.023	0.042	0.001	0.134	#13	0.102	Not Required	Pass
12	0.001	0.434	0.044	0.086	0.009	0.478	#13	0.054	Not Required	Pass
13	0.002	0.112	0.042	0.055	0.002	0.135	#13	0.306	Not Required	Pass
14	0.002	0.078	0.044	0.038	0.002	0.107	#13	0.204	Not Required	Pass
15	0.000	0.018	0.003	0.015	0.000	0.021	#13	Not Required	Not Required	Pass
16	0.000	0.013	0.003	0.011	0.000	0.015	#13	Not Required	Not Required	Pass
101	0.045	0.239	0.003	0.009	0.000	0.244	#32	0.382	Not Required	Pass
102	0.000	0.497	0.052	0.102	0.010	0.549	#13	0.036	Not Required	Pass
103	0.001	0.824	0.006	0.083	0.001	0.830	#13	0.046	Not Required	Pass
104	0.001	0.592	0.012	0.059	0.002	0.598	#13	0.082	Not Required	Pass
105	0.001	0.511	0.015	0.082	0.004	0.515	#13	0.076	Not Required	Pass
106	0.001	0.809	0.006	0.081	0.002	0.810	#13	0.046	Not Required	Pass
107	0.001	0.502	0.010	0.080	0.003	0.504	#13	0.076	Not Required	Pass
108	0.002	0.037	0.011	0.033	0.001	0.048	#13	0.102	Not Required	Pass
109	0.001	0.077	0.013	0.001	0.000	0.090	#13	0.206	Not Required	Pass
110	0.001	0.571	0.011	0.057	0.003	0.581	#13	0.082	Not Required	Pass
111	0.001	0.054	0.011	0.047	0.001	0.058	#13	0.102	Not Required	Pass
112	0.000	0.478	0.051	0.099	0.010	0.530	#13	0.054	Not Required	Pass
113	0.002	0.247	0.037	0.064	0.002	0.254	#13	0.306	Not Required	Pass
114	0.003	0.200	0.039	0.046	0.002	0.212	#13	0.306	Not Required	Pass
115	0.003	0.378	0.020	0.051	0.001	0.396	#13	0.507	Not Required	Pass
116	0.004	0.260	0.021	0.037	0.001	0.275	#13	0.507	Not Required	Pass
201	0.043	0.242	0.000	0.009	0.000	0.246	#32	0.382	Not Required	Pass
202	0.000	0.466	0.049	0.096	0.010	0.515	#13	0.036	Not Required	Pass
203	0.001	0.789	0.004	0.079	0.001	0.793	#13	0.046	Not Required	Pass
204	0.001	0.547	0.010	0.055	0.002	0.553	#13	0.082	Not Required	Pass
205	0.001	0.490	0.010	0.078	0.003	0.492	#13	0.076	Not Required	Pass
206	0.001	0.789	0.004	0.079	0.001	0.793	#13	0.046	Not Required	Pass
207	0.001	0.490	0.010	0.078	0.003	0.492	#13	0.076	Not Required	Pass
208	0.002	0.040	0.011	0.032	0.001	0.044	#13	0.102	Not Required	Pass
209	0.001	0.071	0.009	0.001	0.000	0.081	#13	0.206	Not Required	Pass
210	0.001	0.547	0.010	0.055	0.002	0.553	#13	0.082	Not Required	Pass
211	0.001	0.056	0.011	0.046	0.001	0.060	#13	0.102	Not Required	Pass
212	0.000	0.466	0.049	0.096	0.010	0.515	#13	0.036	Not Required	Pass
213	0.002	0.242	0.028	0.060	0.002	0.247	#13	0.306	Not Required	Pass
214	0.003	0.177	0.028	0.041	0.002	0.184	#13	0.306	Not Required	Pass
215	0.003	0.277	0.016	0.046	0.001	0.289	#13	0.507	Not Required	Pass
216	0.003	0.188	0.016	0.032	0.001	0.203	#13	0.507	Not Required	Pass
301	0.045	0.239	0.003	0.009	0.000	0.244	#32	0.382	Not Required	Pass
302	0.000	0.478	0.051	0.099	0.010	0.530	#13	0.054	Not Required	Pass
303	0.001	0.809	0.006	0.081	0.002	0.810	#13	0.046	Not Required	Pass
304	0.001	0.571	0.011	0.057	0.003	0.581	#13	0.082	Not Required	Pass
305	0.001	0.502	0.010	0.080	0.003	0.504	#13	0.076	Not Required	Pass
306	0.001	0.824	0.006	0.083	0.001	0.830	#13	0.046	Not Required	Pass
307	0.001	0.511	0.015	0.082	0.004	0.515	#13	0.076	Not Required	Pass
308	0.002	0.049	0.021	0.037	0.001	0.054	#13	0.102	Not Required	Pass
309	0.001	0.077	0.013	0.001	0.000	0.090	#13	0.206	Not Required	Pass
310	0.001	0.592	0.012	0.059	0.002	0.598	#13	0.082	Not Required	Pass
311	0.002	0.081	0.020	0.051	0.001	0.086	#13	0.102	Not Required	Pass
312	0.000	0.497	0.052	0.102	0.010	0.549	#13	0.036	Not Required	Pass
313	0.002	0.247	0.037	0.064	0.002	0.251	#13	0.306	Not Required	Pass
314	0.003	0.200	0.039	0.046	0.002	0.212	#13	0.306	Not Required	Pass

315	0.003	0.272	0.016	0.047	0.001	0.289	#13	0.507	Not Required	Pass
316	0.003	0.186	0.016	0.033	0.001	0.199	#13	0.507	Not Required	Pass
401	0.033	0.189	0.030	0.007	0.003	0.195	#32	0.382	Not Required	Pass
402	0.001	0.434	0.044	0.086	0.009	0.478	#13	0.054	Not Required	Pass
403	0.001	0.688	0.022	0.070	0.005	0.710	#13	0.046	Not Required	Pass
404	0.001	0.483	0.020	0.049	0.005	0.503	#13	0.082	Not Required	Pass
405	0.001	0.427	0.015	0.068	0.005	0.437	#13	0.076	Not Required	Pass
406	0.001	0.520	0.014	0.051	0.003	0.534	#13	0.046	Not Required	Pass
407	0.001	0.322	0.009	0.052	0.003	0.331	#13	0.076	Not Required	Pass
408	0.000	0.013	0.003	0.011	0.000	0.015	#13	Not Required	Not Required	Pass
409	0.001	0.080	0.025	0.003	0.002	0.104	#13	0.206	Not Required	Pass
410	0.000	0.352	0.025	0.035	0.005	0.377	#13	0.122	Not Required	Pass
411	0.000	0.018	0.003	0.015	0.000	0.021	#13	Not Required	Not Required	Pass
412	0.001	0.272	0.031	0.062	0.006	0.303	#13	0.054	Not Required	Pass
413	0.002	0.112	0.042	0.055	0.002	0.135	#13	0.204	Not Required	Pass
414	0.002	0.078	0.044	0.038	0.002	0.107	#13	0.306	Not Required	Pass
415	0.003	0.402	0.023	0.042	0.001	0.416	#13	0.507	Not Required	Pass
416	0.004	0.279	0.025	0.028	0.001	0.297	#13	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																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.667</td> <td>11.691</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.663</td> <td>-1.114</td> </tr> <tr> <td>V_z (kip)</td> <td>0.331</td> <td>0.518</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.873</td> <td>1.362</td> </tr> <tr> <td>M_z (kipft)</td> <td>16.643</td> <td>27.852</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_n) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	P (kip)	7.667	11.691	V_x (kip)	-0.663	-1.114	V_z (kip)	0.331	0.518	M_x (kipft)	0.873	1.362	M_z (kipft)	16.643	27.852	
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	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.663 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.10557 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(16.643 \text{ kipft}) + ((-0.663 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6502 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6092 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.331 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.052707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.873 \text{ kipft}) + ((0.331 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13901 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.6988 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6092 \text{ ft}), (2.6988 \text{ ft})]$$

$$L_{e,req} = 5.609 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.609 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.97548$$

Status: **PASS**
Ratio: **0.980**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.667 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47919 \text{ kip/ft}^2$$

$$q = 0.47919 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23959$$

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

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:

$H_o = -0.10557 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6502 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6502 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.6502 \text{ kipft/ft})) + (4 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8968 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6502 \text{ kipft/ft})) + (3 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.6502 \text{ kipft/ft})) + (2 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.25956 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6502 \text{ kipft/ft})) + ((-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.85171 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8968 \text{ ft})}{2}$$

$$p_a = 0.29226 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25956 \text{ kip/ft}^2)}{(0.29226 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88811$$

p_a - Allowable lateral soil pressure at depth L_e ,

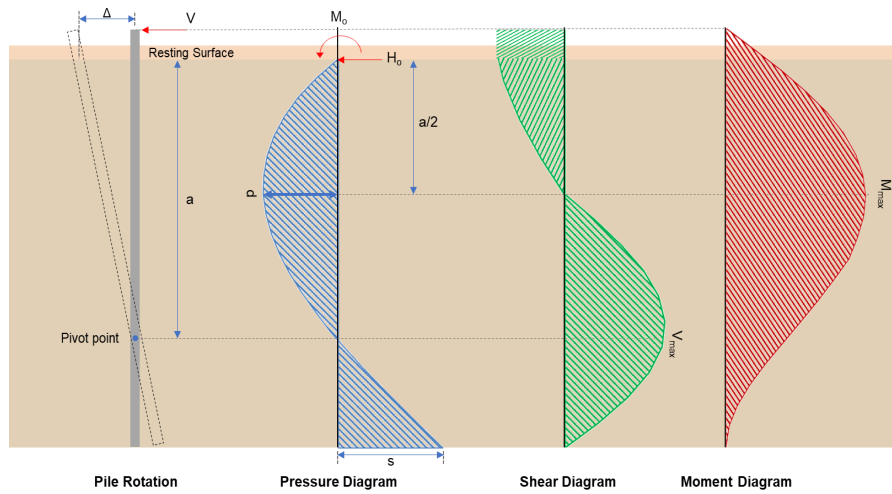
Status: **PASS**
Ratio: **0.890**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85171 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98749$	Status: PASS Ratio: 0.990
	<p>Considering z-direction:</p> <p>$H_o = 0.052707 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.13901 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.13901 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.052707 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.13901 \text{ kipft/ft})) + (4 \times (0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1172 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.13901 \text{ kipft/ft})) + (3 \times (0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.13901 \text{ kipft/ft})) + (2 \times (0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = 0.047599 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.13901 \text{ kipft/ft})) + ((0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = 0.10545 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1172 \text{ ft})}{2}$ $p_a = 0.30879 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.047599 \text{ kip/ft}^2)}{(0.30879 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.15415$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: PASS Ratio: 0.150

$$\text{Ratio} = \frac{(0.10545 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.12226$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.114 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.17739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.852 \text{ kipft}) + ((-1.114 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.435 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.435 \text{ kipft/ft})}{(-0.17739 \text{ kip/ft})}$$

$$E = 25.002 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.435 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.17739 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.435 \text{ kipft/ft})) + (4 \times (-0.17739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.897 \text{ ft}$$

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

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

$$V_{max} = ((-0.17739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.897 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.897 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.9369 \text{ kip}$$

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

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

$$M_{max} = ((-0.17739 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(25.002 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 16.694 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.518 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.082484 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.362 \text{ kipft}) + ((0.518 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21688 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21688 \text{ kipft/ft})}{(0.082484 \text{ kip/ft})}$$

$$E = 2.6293 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21688 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (0.082484 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.21688 \text{ kipft/ft})) + (4 \times (0.082484 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1176 \text{ ft}$$

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

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

$$V_{max} = ((0.082484 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1176 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1176 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.4871 \text{ kip}$$

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

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

$$M_{max} = ((0.082484 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.6293 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.2314 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.691 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.87 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.87 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3 s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.691 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0036725$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 11.691 \text{ kip} \rightarrow 11691 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(11691 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.35 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

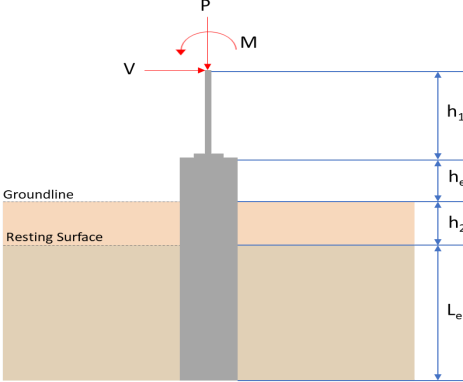
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.35 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.35 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.46 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 5.9369 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(5.9369 \text{ kip})}{(118.46 \text{ kip})}$ $\text{Ratio} = 0.050117$ <p>Considering z-direction:</p> <p>$V_{max} = 0.4871 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.4871 \text{ kip})}{(118.46 \text{ kip})}$ $\text{Ratio} = 0.0041119$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 16.694\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(16.694\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.061057$	<p>Status: PASS Ratio: 0.060</p>
	<p>Considering z-direction: $M_{max} = 1.2314\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2314\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0045036$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5.75$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.667</td> <td>11.691</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.663</td> <td>-1.114</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.331</td> <td>-0.518</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.873</td> <td>-1.362</td> </tr> <tr> <td>M_z (kipft)</td> <td>16.643</td> <td>27.852</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	7.667	11.691	V_x (kip)	-0.663	-1.114	V_z (kip)	-0.331	-0.518	M_x (kipft)	-0.873	-1.362	M_z (kipft)	16.643	27.852	
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M_z (kipft)	16.643	27.852																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.663 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.10557 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(16.643 \text{ kipft}) + ((-0.663 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.6502 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.6092 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.331 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.052707 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.873 \text{ kipft}) + ((-0.331 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.13901 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7682 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6092 \text{ ft}), (1.7682 \text{ ft})]$$

$$L_{e,req} = 5.609 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(5.609 \text{ ft})}{(5.75 \text{ ft})}$$

$$\text{Ratio} = 0.97548$$

Status: **PASS**
Ratio: **0.980**

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.667 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47919 \text{ kip/ft}^2$$

$$q = 0.47919 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23959$$

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

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:

$H_o = -0.10557 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.6502 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.6502 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (2.6502 \text{ kipft/ft})) + (4 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.8968 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.6502 \text{ kipft/ft})) + (3 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 \times [(3 \times (2.6502 \text{ kipft/ft})) + (2 \times (-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$$

$$p = 0.25956 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.6502 \text{ kipft/ft})) + ((-0.10557 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$$

$$s = 0.85171 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.8968 \text{ ft})}{2}$$

$$p_a = 0.29226 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25956 \text{ kip/ft}^2)}{(0.29226 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88811$$

p_a - Allowable lateral soil pressure at depth L_e ,

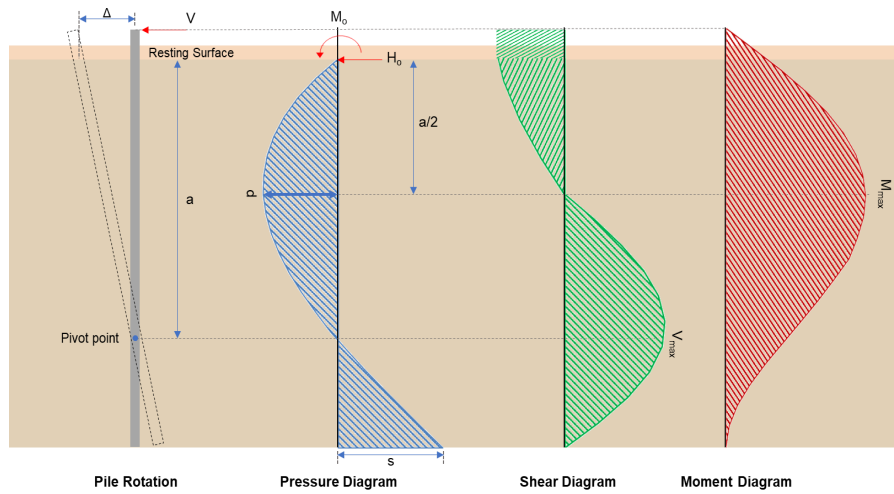
Status: **PASS**
Ratio: **0.890**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.85171 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.98749$	<p>Status: PASS Ratio: 0.990</p>
	<p>Considering z-direction:</p> <p>$H_o = -0.052707 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.13901 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.13901 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.052707 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.13901 \text{ kipft/ft})) + (4 \times (-0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))}$ $a = 4.1172 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.13901 \text{ kipft/ft})) + (3 \times (-0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]^2}{(5.75 \text{ ft})^2 [(3 \times (0.13901 \text{ kipft/ft})) + (2 \times (-0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}$ $p = -0.014961 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.13901 \text{ kipft/ft})) + ((-0.052707 \text{ kip/ft}) \times (5.75 \text{ ft}))]}{(5.75 \text{ ft})^2}$ $s = -0.0045441 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.1172 \text{ ft})}{2}$ $p_a = 0.30879 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(-0.014961 \text{ kip/ft}^2)}{(0.30879 \text{ kip/ft}^2)}$ $\text{Ratio} = -0.04845$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5.75 \text{ ft})$ $p_s = 0.8625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	<p>Status: PASS Ratio: -0.050</p>

$$Ratio = \frac{(-0.0045441 \text{ kip/ft}^2)}{(0.8625 \text{ kip/ft}^2)}$$

$$Ratio = -0.0052685$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.114 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.17739 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(27.852 \text{ kipft}) + ((-1.114 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.435 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.435 \text{ kipft/ft})}{(-0.17739 \text{ kip/ft})}$$

$$E = 25.002 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.435 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.17739 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (4.435 \text{ kipft/ft})) + (4 \times (-0.17739 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 3.897 \text{ ft}$$

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

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

$$V_{max} = ((-0.17739 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.897 \text{ ft})}{(5.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.897 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 5.9369 \text{ kip}$$

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

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

$$M_{max} = ((-0.17739 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(25.002 \text{ ft})}{(5.75 \text{ ft})} + \frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (25.002 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(3.897 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 16.694 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.518 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.082484 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.362 \text{ kipft}) + ((-0.518 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.21688 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.21688 \text{ kipft/ft})}{(-0.082484 \text{ kip/ft})}$$

$$E = 2.6293 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.21688 \text{ kipft/ft}) \times (5.75 \text{ ft})) + (3 \times (-0.082484 \text{ kip/ft}) \times (5.75 \text{ ft})^2)}{(6 \times (0.21688 \text{ kipft/ft})) + (4 \times (-0.082484 \text{ kip/ft}) \times (5.75 \text{ ft}))}$$

$$a = 4.1176 \text{ ft}$$

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

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

$$V_{max} = ((-0.082484 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1176 \text{ ft})}{(5.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1176 \text{ ft})}{(5.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.4871 \text{ kip}$$

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

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

$$M_{max} = ((-0.082484 \text{ kip/ft}) \times (48 \text{ in}) \times (5.75 \text{ ft})) \times \left[\left(\frac{(2.6293 \text{ ft})}{(5.75 \text{ ft})} + \frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (2.6293 \text{ ft})}{(5.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.1176 \text{ ft})}{2 \times (5.75 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 1.2314 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.691 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.87 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.87 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.691 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0036725$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 11.691 \text{ kip} \rightarrow 11691 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(11691 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.35 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

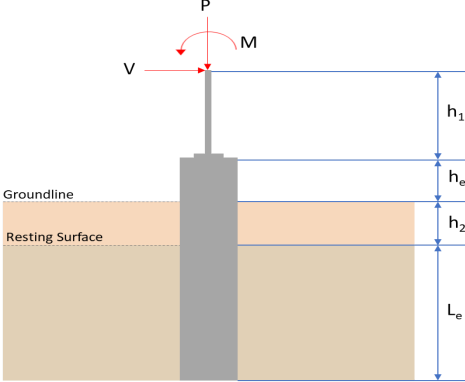
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.35 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.35 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.35 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.46 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 5.9369 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(5.9369 \text{ kip})}{(118.46 \text{ kip})}$ $\text{Ratio} = 0.050117$ <p>Considering z-direction:</p> <p>$V_{max} = 0.4871 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.4871 \text{ kip})}{(118.46 \text{ kip})}$ $\text{Ratio} = 0.0041119$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 16.694\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(16.694\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.061057$	<p>Status: PASS Ratio: 0.060</p>
	<p>Considering z-direction: $M_{max} = 1.2314\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(1.2314\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0045036$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.255</td> <td>15.726</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.896</td> <td>-1.489</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.040</td> <td>-0.062</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.123</td> <td>-0.191</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.085</td> <td>35.332</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	10.255	15.726	V_x (kip)	-0.896	-1.489	V_z (kip)	-0.040	-0.062	M_x (kipft)	-0.123	-0.191	M_z (kipft)	21.085	35.332	
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M_x (kipft)	-0.123	-0.191																										
M_z (kipft)	21.085	35.332																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.896 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.14268 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.085 \text{ kipft}) + ((-0.896 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3575 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.0106 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.04 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0063694 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((-0.04 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.019586 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0522 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.0106 \text{ ft}), (1.0522 \text{ ft})]$$

$$L_{e,req} = 6.011 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.011 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.96176$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(10.255 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.64094 \text{ kip/ft}^2$$

$$q = 0.04094 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.64094 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32047$$

Status: **PASS**
Ratio: **0.320**

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:

$H_o = -0.14268 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3575 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3575 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3575 \text{ kipft/ft})) + (4 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.245 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3575 \text{ kipft/ft})) + (3 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3575 \text{ kipft/ft})) + (2 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.26792 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3575 \text{ kipft/ft})) + ((-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89445 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.245 \text{ ft})}{2}$$

$$p_a = 0.31838 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26792 \text{ kip/ft}^2)}{(0.31838 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84151$$

p_s - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.840**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89445 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95408$$

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

Considering z-direction:

$H_o = -0.0063694 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.019586 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019586 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.019586 \text{ kipft/ft})) + (4 \times (-0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4663 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.019586 \text{ kipft/ft})) + (3 \times (-0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.019586 \text{ kipft/ft})) + (2 \times (-0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0015535 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.019586 \text{ kipft/ft})) + ((-0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.000097834 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4663 \text{ ft})}{2}$$

$$p_a = 0.33498 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0015535 \text{ kip/ft}^2)}{(0.33498 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.0046376$$

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

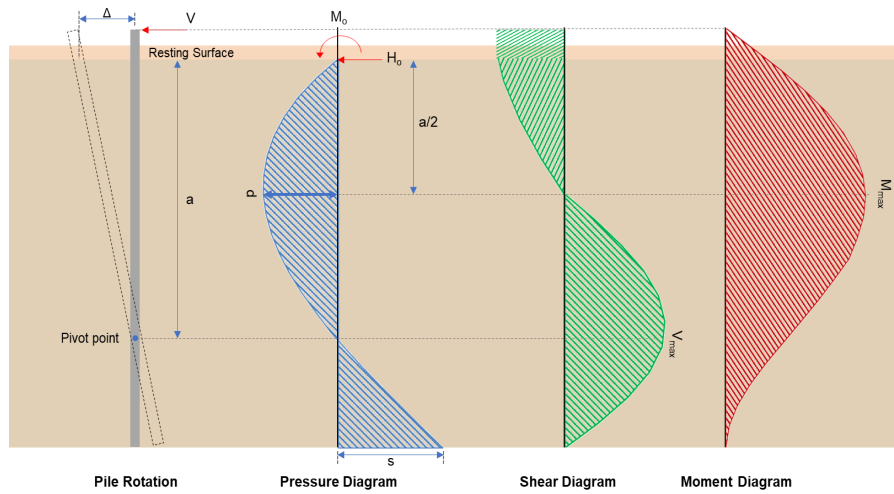
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(-0.000097834 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = -0.00010436$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.489 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.2371 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.332 \text{ kipft}) + ((-1.489 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6261 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6261 \text{ kipft/ft})}{(-0.2371 \text{ kip/ft})}$$

$$E = 23.729 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6261 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.2371 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.6261 \text{ kipft/ft})) + (4 \times (-0.2371 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2445 \text{ ft}$$

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

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

$$V_{max} = ((-0.2371 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2445 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2445 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0063 \text{ kip}$$

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

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

$$M_{max} = ((-0.2371 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(23.729 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 21.352 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.062 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0098726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.191 \text{ kipft}) + ((-0.062 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.030414 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.030414 \text{ kipft/ft})}{(-0.0098726 \text{ kip/ft})}$$

$$E = 3.0806 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.030414 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.0098726 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.030414 \text{ kipft/ft})) + (4 \times (-0.0098726 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4661 \text{ ft}$$

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

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

$$V_{max} = ((-0.0098726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4661 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4661 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.06076 \text{ kip}$$

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

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

$$M_{max} = ((-0.0098726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.0806 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.16787 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.726 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.74 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.74 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.726 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.00494$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1 The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 15.726 \text{ kip} \rightarrow 15726 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(15726 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.89 \text{ kip}$$

22.5.5.1.2 The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

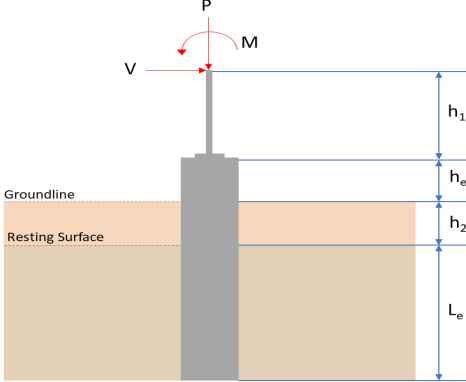
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.89 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.89 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ytik} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.89 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.81 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.0063 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.0063 \text{ kip})}{(118.81 \text{ kip})}$ $\text{Ratio} = 0.058971$ <p>Considering z-direction:</p> <p>$V_{max} = 0.06076 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.06076 \text{ kip})}{(118.81 \text{ kip})}$ $\text{Ratio} = 0.00051141$	<p>Status: PASS Ratio: 0.060</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 21.352\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(21.352\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.078091$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 0.16787\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.16787\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00061394$	<p>Status: PASS Ratio: 0.000</p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>9.829</td> <td>15.062</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.859</td> <td>-1.433</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.319</td> <td>35.743</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	9.829	15.062	V_x (kip)	-0.859	-1.433	V_z (kip)	0.000	0.000	M_x (kipft)	0.000	0.000	M_z (kipft)	21.319	35.743	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_x (kipft)	0.000	0.000																										
M_z (kipft)	21.319	35.743																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.859 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.13678 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.319 \text{ kipft}) + ((-0.859 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3947 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$$L_{e,x} = 6.0541 \text{ ft} - \text{Required depth in x-direction,}$$

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.0541 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 6.054 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.054 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.96864$$

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

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

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

$$q = \frac{(9.829 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.61431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.61431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.30716$$

Status: **PASS**
Ratio: **0.310**

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:

$H_o = -0.13678$ kip/ft - Lateral force per length of pile,

$M_o = 3.3947$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3947 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.13678 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3947 \text{ kipft/ft})) + (4 \times (-0.13678 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2415 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3947 \text{ kipft/ft})) + (3 \times (-0.13678 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3947 \text{ kipft/ft})) + (2 \times (-0.13678 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.27485 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3947 \text{ kipft/ft})) + ((-0.13678 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.91155 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2415 \text{ ft})}{2}$$

$$p_a = 0.31812 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27485 \text{ kip/ft}^2)}{(0.31812 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.86401$$

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

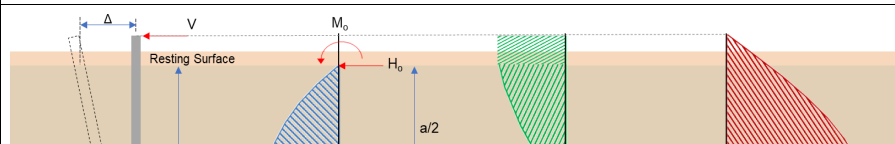
$$\text{Ratio} = \frac{s}{p_s}$$

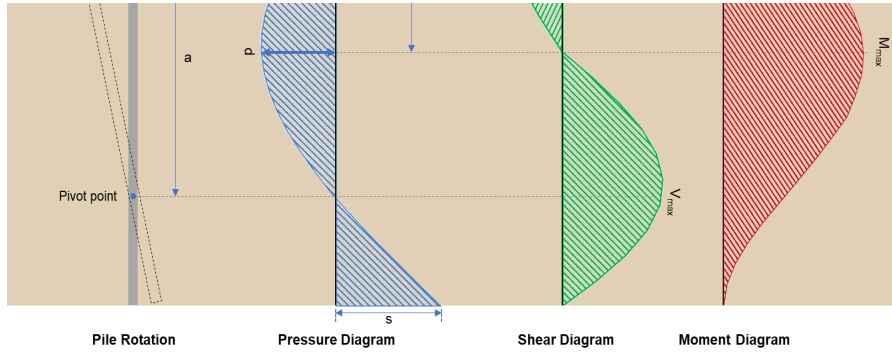
$$\text{Ratio} = \frac{(0.91155 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.97232$$

Status: **PASS**
Ratio: **0.860**

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.433 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.22818 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.743 \text{ kipft}) + ((-1.433 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6916 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6916 \text{ kipft/ft})}{(-0.22818 \text{ kip/ft})}$$

$$E = 24.943 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6916 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.22818 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.6916 \text{ kipft/ft})) + (4 \times (-0.22818 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2412 \text{ ft}$$

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

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

$$V_{max} = ((-0.22818 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (24.943 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2412 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (24.943 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2412 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0577 \text{ kip}$$

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

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

$$M_{max} = ((-0.22818 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(24.943 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2412 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (24.943 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2412 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (24.943 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2412 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.533 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.062 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.76 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.76 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Axial Compression Strength (ACI 318-19, LRFD)22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.062 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0047314$$

Status: **PASS**
Ratio: **0.000****Shear Strength (ACI 318-19, LRFD)****Parameters:** $b_w = 48 \text{ in}$ - Effective width,22.5.2.2 d - Effective depth

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.1 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 15.062 \text{ kip} \rightarrow 15062 \text{ lbf}$,22.5.5.1.1(a) $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(15062 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.8 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

 V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.8 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.8 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.8 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.75 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.0577 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.0577 \text{ kip})}{(118.75 \text{ kip})}$ $\text{Ratio} = 0.059432$	<p>Status: PASS Ratio: 0.060</p>
<p>14.5.2.1b</p>	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kip ft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = 0.85 f'_c S_m$	

$$\phi M_{n,z} = \phi S_x F_y$$

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

$$\phi M_{n,z} = 2545.9 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$$

$$\phi M_n = 273.42 \text{ kipft}$$

Considering x-direction:

$M_{max} = 21.533 \text{ kipft}$ - Maximum moment in the x-direction,

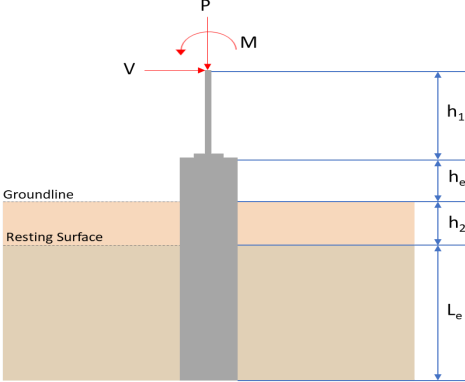
Ratio - Capacity

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

$$\text{Ratio} = \frac{(21.533 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.078752$$

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

REFERENCES	CALCULATIONS	RESULTS																										
	<p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>10.255</td> <td>15.726</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.896</td> <td>-1.489</td> </tr> <tr> <td>V_z (kip)</td> <td>0.040</td> <td>0.062</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.123</td> <td>0.191</td> </tr> <tr> <td>M_z (kipft)</td> <td>21.085</td> <td>35.332</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ ksi - Concrete strength,</p>	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	Load Component	ASD	LRFD	P (kip)	10.255	15.726	V_x (kip)	-0.896	-1.489	V_z (kip)	0.040	0.062	M_x (kipft)	0.123	0.191	M_z (kipft)	21.085	35.332	
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M_z (kipft)	21.085	35.332																										
	<p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.896 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.14268 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(21.085 \text{ kipft}) + ((-0.896 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.3575 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.0106 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.04 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0063694 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.123 \text{ kipft}) + ((0.04 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.019586 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.2709 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.0106 \text{ ft}), (1.2709 \text{ ft})]$$

$$L_{e,req} = 6.011 \text{ ft}$$

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

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

$$\text{Ratio} = \frac{(6.011 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.96176$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(10.255 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.64094 \text{ kip/ft}^2$$

$$q = 0.04094 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.64094 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.32047$$

Status: **PASS**
Ratio: **0.320**

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:

$H_o = -0.14268 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.3575 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.3575 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.3575 \text{ kipft/ft})) + (4 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.245 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.3575 \text{ kipft/ft})) + (3 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.3575 \text{ kipft/ft})) + (2 \times (-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.26792 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.3575 \text{ kipft/ft})) + ((-0.14268 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89445 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.245 \text{ ft})}{2}$$

$$p_a = 0.31838 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.26792 \text{ kip/ft}^2)}{(0.31838 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.84151$$

p_s - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.840**

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89445 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.95408$$

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

Considering z-direction:

$H_o = 0.0063694 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.019586 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.019586 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.019586 \text{ kipft/ft})) + (4 \times (0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4663 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.019586 \text{ kipft/ft})) + (3 \times (0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.019586 \text{ kipft/ft})) + (2 \times (0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.0054271 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.019586 \text{ kipft/ft})) + ((0.0063694 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.012131 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4663 \text{ ft})}{2}$$

$$p_a = 0.33498 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0054271 \text{ kip/ft}^2)}{(0.33498 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.016201$$

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

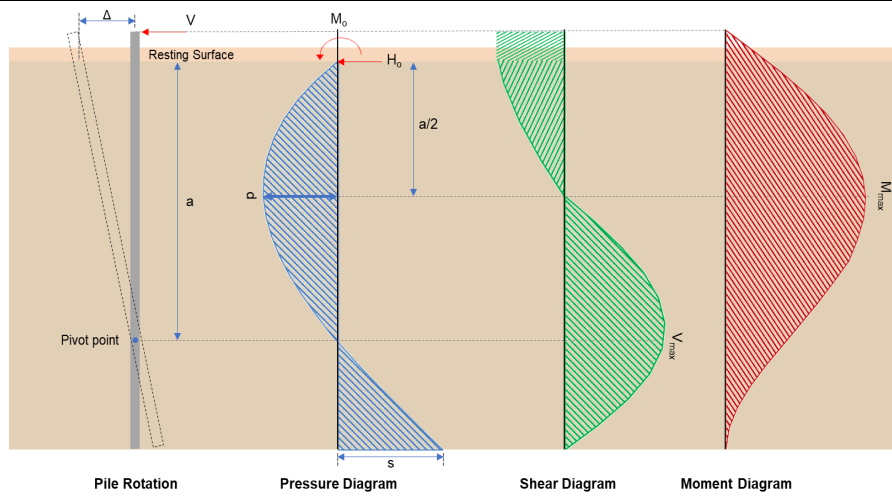
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$\text{Ratio} = \frac{(0.012131 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.01294$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.489 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.2371 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(35.332 \text{ kipft}) + ((-1.489 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.6261 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.6261 \text{ kipft/ft})}{(-0.2371 \text{ kip/ft})}$$

$$E = 23.729 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.6261 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.2371 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.6261 \text{ kipft/ft})) + (4 \times (-0.2371 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2445 \text{ ft}$$

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

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

$$V_{max} = ((-0.2371 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2445 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2445 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 7.0063 \text{ kip}$$

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

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

$$M_{max} = ((-0.2371 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(23.729 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.729 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2445 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 21.352 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.062 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0098726 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.191 \text{ kipft}) + ((0.062 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.030414 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.030414 \text{ kipft/ft})}{(0.0098726 \text{ kip/ft})}$$

$$E = 3.0806 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.030414 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.0098726 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.030414 \text{ kipft/ft})) + (4 \times (0.0098726 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4661 \text{ ft}$$

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

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

$$V_{max} = ((0.0098726 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4661 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4661 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.06076 \text{ kip}$$

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

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

$$M_{max} = ((0.0098726 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.0806 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.0806 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4661 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.16787 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.8$ - Alpha factor for axial strength,
 $A_g = 2304 \text{ in}^2$ - Gross area of concrete,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(15.726 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.74 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.74 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 3183.4 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(15.726 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.00494$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,max} = 324.49 \text{ kip}$$

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 15.726 \text{ kip} \rightarrow 15726 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(15726 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.89 \text{ kip}$$

22.5.5.1.2

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$,
 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,b} = 406.27 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.89 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.89 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.89 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.81 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 7.0063 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(7.0063 \text{ kip})}{(118.81 \text{ kip})}$ $\text{Ratio} = 0.058971$ <p>Considering z-direction:</p> <p>$V_{max} = 0.06076 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.06076 \text{ kip})}{(118.81 \text{ kip})}$ $\text{Ratio} = 0.00051141$	<p>Status: PASS Ratio: 0.060</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p>Considering x-direction: $M_{max} = 21.352\text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(21.352\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.078091$	<p>Status: PASS Ratio: 0.080</p>
	<p>Considering z-direction: $M_{max} = 0.16787\text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.16787\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00061394$	<p>Status: PASS Ratio: 0.000</p>