

Your Project Calculations



Project Name: Beehive11a-JB-RevA

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=Beehive11a-JB-RevA&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=YGy4QJCCzdgIbX88hBSYaoDmqoUTKvacduqjmqwaYWgKg7BsnA7L81fy3ijebLT

Array Specification

Product:	Beam
Unique ID:	4P-19.75-10TOP-HD-45-L-5Hx10W-CK6K
Duty Classification:	HD
Module Width:	41.10 in
Module Length:	87.20in
Number of Rows:	5
Number of Columns:	10
Total Number of Modules:	50
Desired Tilt Angle:	46
Front Edge Clearance:	6
Total Array Height at Tilt:	18.39 ft
Total Frame Length:	74.25 ft
Frame Weight:	4326 lbs
Array Dimensions N/S:	17.33 ft
Array Dimensions E/W:	73.50 ft
Rail Length:	208.00 in
Rail Spacing:	3.68 ft
Rail Check:	FAIL (104% utilized)

Support Specifications

Pole Size:	10in Pipe Sch 40
Pole Length above Grade:	12.23 ft
Number of Poles:	4
Pole Spacing:	19.75 ft

Foundation Specifications

Foundation Type:	Round
Foundation Dimensions:	Ø96 in
Foundation Depth (below grade):	Pile 1: 6.25 ft Pile 2: 6.50 ft Pile 3: 6.50 ft Pile 4: 6.25 ft
Foundation Volume:	47.473 y ³
Foundation Result:	PASSED
Mount Twist:	0.613735 kip

Site Info

Risk Category:	I
Exposure:	C
Soil Classification:	sandy_gravel
Site Location:	Big Sky, MT, USA
Wind Speed:	105 mph
Snow Load:	80 psf
Design Uplift Pressure:	0.017379 ksf
Design Downforce Pressure:	-0.017379 ksf
Design Snow Pressure:	0.021113 ksf



Design Disclaimer

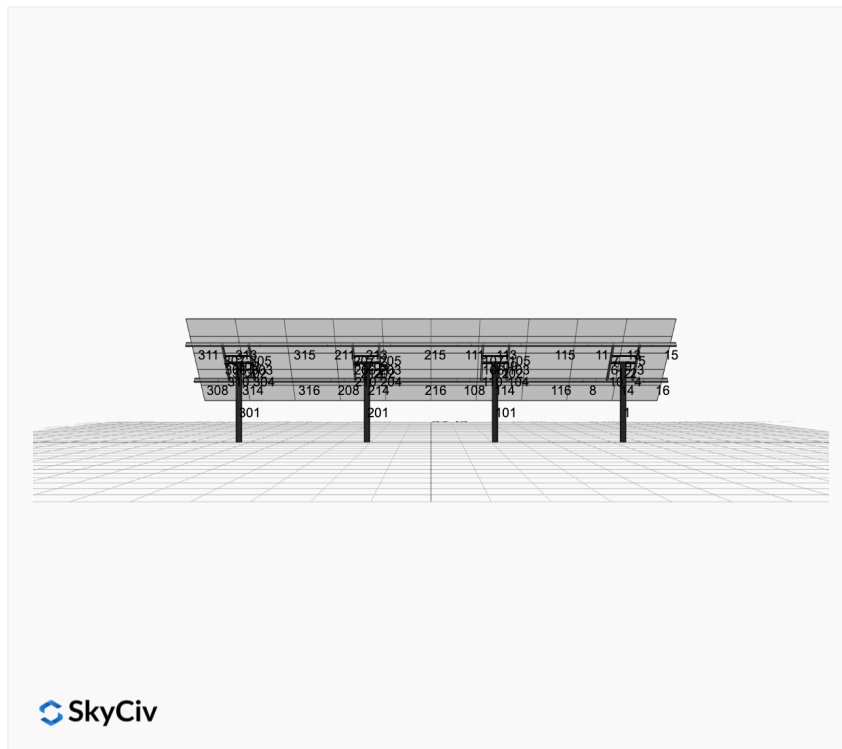
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

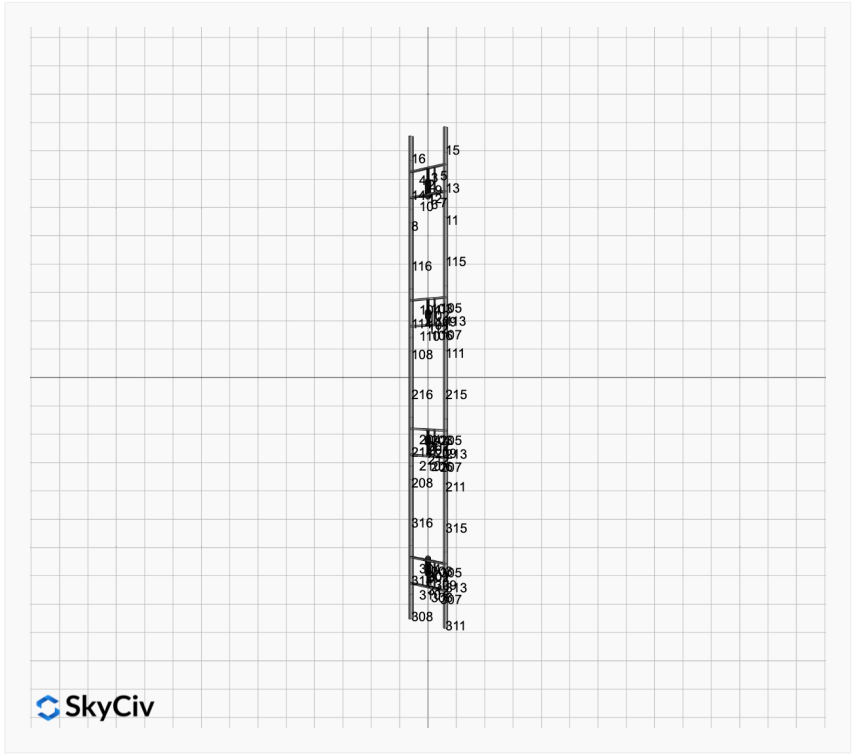
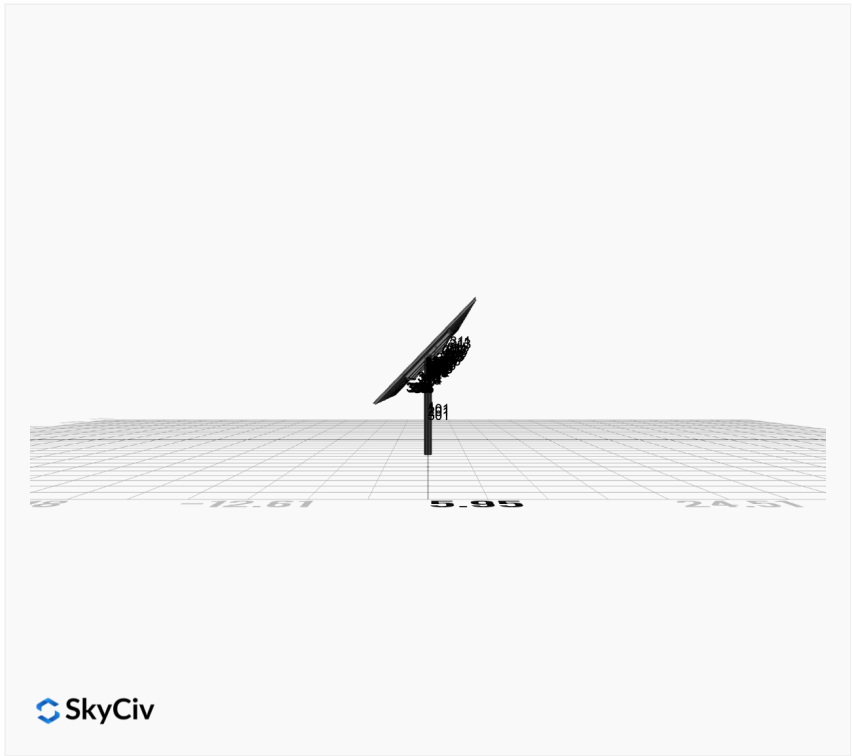
AutoDesigner Input

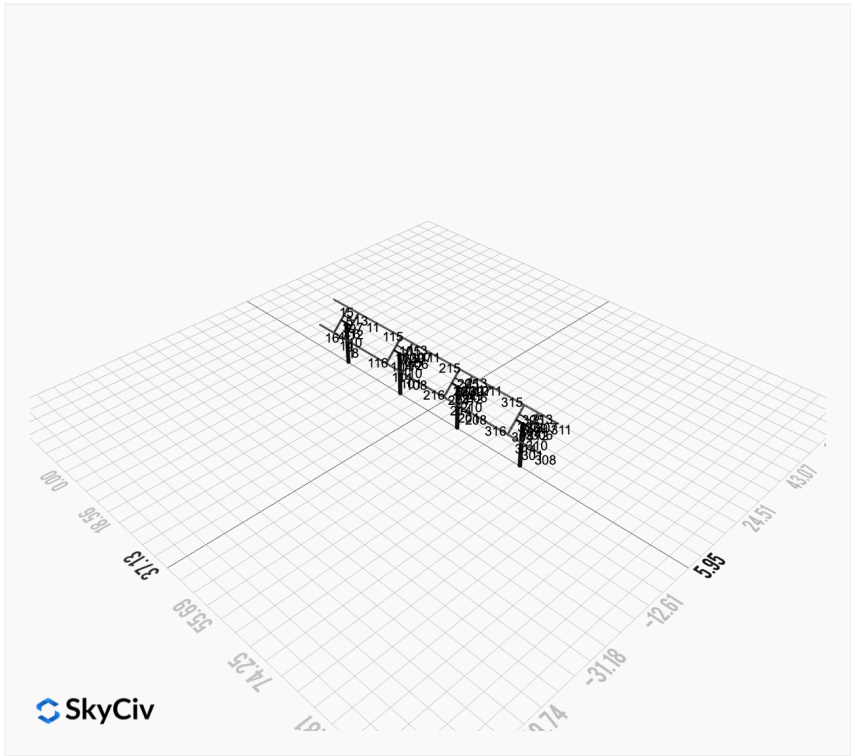
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Design Notes:

- AISC Deflection checks are set to L/1 due to structure design intent







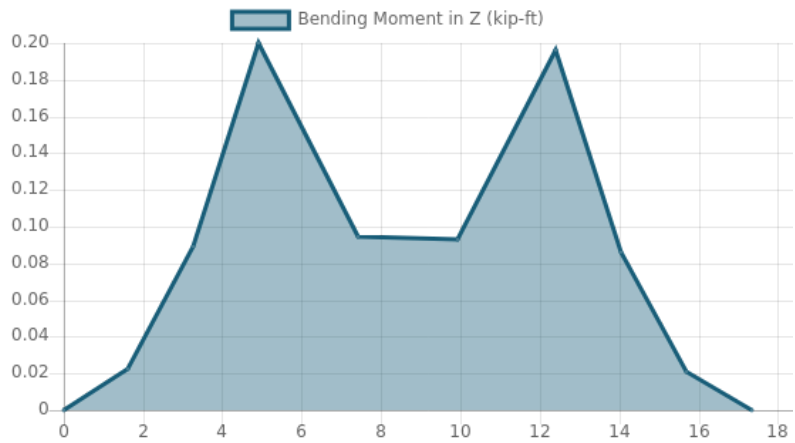
Rail Design Check

Rail Length: 17.333333333333332 ft
Additional Restraints Required: None
Tributary Width: 3.6750000000000003 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0539 kip/ft
Snow (Y): -0.0558 kip/ft
Wind uplift Case A: 0.0639 kip/ft
Wind downforce Case A: 0.0639 kip/ft
Dead (Panel load) (X): 0.0114 kip/ft
Dead (Panel load) (Y): -0.0118 kip/ft



Result Check	Max Limit	Max Value	Utility	Status
Custom Stress Limit	34.5	35.97598376	1.043	FAIL
Material Yield	34.5	35.97598376	1.043	FAIL
Material Strength	37	35.97598376	0.972	PASS

Member 1, ULS: 1.14D



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	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 2. D + L	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 3. D + (S or Lr or R)	0.0303	6.7756	0.1078	0.3656	-0.0199	-0.3141
ULS: 3. D + (S or Lr or R)	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0248	5.6771	0.0882	0.2992	-0.0162	-0.2547
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 5b. D + 0.7E	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0248	5.6771	0.0882	0.2992	-0.0162	-0.2547
ULS: 8. 0.6D + 0.7E	0.0050	1.4290	0.0177	0.0600	-0.0030	-0.0461
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2229	4.5244	0.0845	0.2610	-0.3632	27.4267
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2390	0.2392	-0.0248	-0.0593	0.3497	-27.2709
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6486	7.2842	0.1294	0.4200	-0.2848	20.3729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0248	5.6771	0.0882	0.2992	-0.0162	-0.2547
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6978	4.0703	0.0475	0.1797	0.2499	-20.6503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0248	5.6771	0.0882	0.2992	-0.0162	-0.2547
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6651	3.9887	0.0708	0.2207	-0.2736	20.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6814	0.7748	-0.0112	-0.0195	0.2610	-20.4724
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0083	2.3817	0.0296	0.0999	-0.0050	-0.0768
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2262	3.5718	0.0727	0.2210	-0.3612	27.4574
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0050	1.4290	0.0177	0.0600	-0.0030	-0.0461
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2357	-0.7135	-0.0366	-0.0993	0.3517	-27.2402
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0050	1.4290	0.0177	0.0600	-0.0030	-0.0461

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	11.6742
Shear X	-3.7394
Shear Z	0.2078
Moment X	0.6869
Moment Y (Twist)	0.6134
Moment Z	46.0511

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	7.2842
Shear X	-2.2390
Shear Z	0.1294
Moment X	0.4200
Moment Y (Twist)	0.3632
Moment Z	27.4574

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	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 2. D + L	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 3. D + (S or Lr or R)	-0.0303	7.6731	-0.0034	-0.0119	0.0179	0.4178
ULS: 3. D + (S or Lr or R)	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0248	6.4122	-0.0028	-0.0097	0.0146	0.3439
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 5b. D + 0.7E	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0248	6.4122	-0.0028	-0.0097	0.0146	0.3439
ULS: 8. 0.6D + 0.7E	-0.0050	1.5775	-0.0006	-0.0020	0.0030	0.0733
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5552	5.1007	0.0051	0.0129	-0.0500	31.4266
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5391	0.1575	-0.0067	-0.0186	0.0576	-30.7954
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9350	8.2658	0.0017	0.0024	-0.0265	23.8221
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0248	6.4122	-0.0028	-0.0097	0.0146	0.3439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8858	4.5583	-0.0071	-0.0212	0.0542	-22.8443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0248	6.4122	-0.0028	-0.0097	0.0146	0.3439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9185	4.4828	0.0036	0.0089	-0.0362	23.6005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9023	0.7754	-0.0052	-0.0148	0.0444	-23.0660
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0083	2.6292	-0.0009	-0.0033	0.0049	0.1222
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5519	4.0490	0.0055	0.0142	-0.0519	31.3777
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0050	1.5775	-0.0006	-0.0020	0.0030	0.0733
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5424	-0.8942	-0.0063	-0.0173	0.0556	-30.8443
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0050	1.5775	-0.0006	-0.0020	0.0030	0.0733

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2846
Shear X	-4.2652
Shear Z	-0.0122
Moment X	-0.0345
Moment Y (Twist)	0.1027
Moment Z	52.9924

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.2658
Shear X	-2.5552
Shear Z	-0.0071
Moment X	-0.0212
Moment Y (Twist)	0.0576
Moment Z	31.4266

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

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 2. D + L	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 3. D + (S or Lr or R)	-0.0303	7.6731	0.0034	0.0119	-0.0178	0.4178
ULS: 3. D + (S or Lr or R)	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0248	6.4122	0.0028	0.0097	-0.0145	0.3439
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 5b. D + 0.7E	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0248	6.4122	0.0028	0.0097	-0.0145	0.3439
ULS: 8. 0.6D + 0.7E	-0.0050	1.5775	0.0006	0.0020	-0.0029	0.0733
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.5552	5.1007	-0.0051	-0.0129	0.0500	31.4266
ULS: 5a. D + 0.6W_Wind downforce Case B only	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.5391	0.1575	0.0067	0.0186	-0.0576	-30.7954
ULS: 5a. D + 0.6W_Wind uplift Case B only	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.9350	8.2658	-0.0017	-0.0025	0.0266	23.8221
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0248	6.4122	0.0028	0.0097	-0.0145	0.3439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.8858	4.5583	0.0071	0.0212	-0.0541	-22.8443
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0248	6.4122	0.0028	0.0097	-0.0145	0.3439

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	-1.9185	4.4828	-0.0036	-0.0089	0.0363	23.6005
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.9023	0.7754	0.0052	0.0148	-0.0444	-23.0660
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	-0.0083	2.6292	0.0009	0.0033	-0.0049	0.1222
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.5519	4.0490	-0.0055	-0.0142	0.0519	31.3777
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-0.0050	1.5775	0.0006	0.0020	-0.0029	0.0733
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.5424	-0.8942	0.0063	0.0173	-0.0556	-30.8443
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	-0.0050	1.5775	0.0006	0.0020	-0.0029	0.0733

Worst Case Reactions LRFD

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

Result	Value (kip, kip-ft)
Axial	13.2846
Shear X	-4.2652
Shear Z	0.0122
Moment X	0.0347
Moment Y (Twist)	0.1026
Moment Z	52.9924

Worst Case Reactions ASD

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

Result	Value (kip, kip-ft)
Axial	8.2658
Shear X	-2.5552
Shear Z	0.0071
Moment X	0.0212
Moment Y (Twist)	0.0576
Moment Z	31.4266

Reaction Forces for Foundation 4 (Node ID#301), (kip, kip-ft)

ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 2. D + L	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 3. D + (S or Lr or R)	0.0303	6.7756	-0.1078	-0.3658	0.0200	-0.3140
ULS: 3. D + (S or Lr or R)	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0248	5.6771	-0.0883	-0.2994	0.0162	-0.2547
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 5b. D + 0.7E	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0248	5.6771	-0.0883	-0.2994	0.0162	-0.2547
ULS: 8. 0.6D + 0.7E	0.0050	1.4290	-0.0177	-0.0600	0.0030	-0.0461
ULS: 5a. D + 0.6W_Wind downforce Case A only	-2.2229	4.5244	-0.0845	-0.2610	0.3632	27.4267
ULS: 5a. D + 0.6W_Wind downforce Case B only	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 5a. D + 0.6W_Wind uplift Case A only	2.2390	0.2392	0.0248	0.0593	-0.3497	-27.2709
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6486	7.2842	-0.1295	-0.4202	0.2848	20.3729
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0248	5.6771	-0.0883	-0.2994	0.0162	-0.2547
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6978	4.0703	-0.0475	-0.1799	-0.2498	-20.6503
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0248	5.6771	-0.0883	-0.2994	0.0162	-0.2547
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.6651	3.9887	-0.0708	-0.2207	0.2737	20.5508
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	1.6814	0.7748	0.0112	0.0195	-0.2610	-20.4724
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.0083	2.3817	-0.0296	-0.1000	0.0050	-0.0768
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-2.2262	3.5718	-0.0727	-0.2210	0.3612	27.4574
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	0.0050	1.4290	-0.0177	-0.0600	0.0030	-0.0461
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	2.2357	-0.7135	0.0366	0.0993	-0.3517	-27.2402
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.0050	1.4290	-0.0177	-0.0600	0.0030	-0.0461

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	11.6742
Shear X	-3.7394
Shear Z	-0.2078
Moment X	-0.6875
Moment Y (Twist)	0.6137
Moment Z	46.0517

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	7.2842
Shear X	-2.2390
Shear Z	-0.1295
Moment X	-0.4202
Moment Y (Twist)	0.3632
Moment Z	27.4574

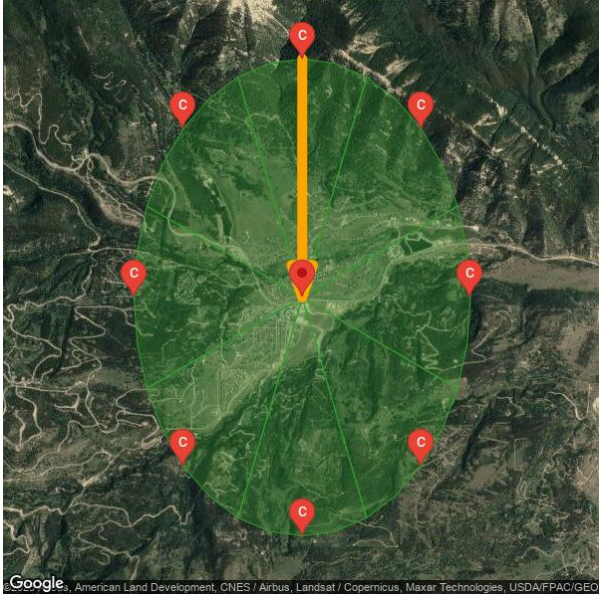

REFERENCES	CALCULATIONS	RESULTS												
Wind Load Calculations based on ASCE 7-16														
<p>Design Information : Project Name : Beehive11a-JB-RevA Client : Designer : MT_SKYCIV AutoDesigner Company : MT Solar Units : Imperial Notes : Wind Loads are based on Freestanding Wall. Wind loads are applied by summing the total individual point loads then taking worst case scenario between Case A and Case C. We then divide this total force by the length of the members and apply as a distributed load. Note: Case C is combined into a single load, then applied as a uniform distributed load.</p>														
<p>Project Data The structure is located in Big Sky, MT, USA categorized as Exposure C (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Solid freestanding walls and attached signs - is based on the Directional Procedure (Chapter 29) of ASCE 7. Moreover, the structure is classified as Risk Category I. The location is elevated at 630 3 ft above mean sea level.</p>  <p style="text-align: center;">Figure 1. Site location.</p> <p>Additional details of the structure are shown in Table below and illustrated in Figure 2:</p> <table border="1" data-bbox="592 1285 1003 1449"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Ground to Top of Wall/Sign, h</td> <td>18.394 ft</td> </tr> <tr> <td>Wall/Sign Horizontal Dimension, B</td> <td>73.500 ft</td> </tr> <tr> <td>Wall/Sign Vertical Dimension, s</td> <td>12.319 ft</td> </tr> <tr> <td>Ratio of Solid Area to Gross Area, ϵ</td> <td>1.000</td> </tr> <tr> <td>Length of return corner, L_r</td> <td>- ft</td> </tr> </tbody> </table>  <p style="text-align: center;">Figure 2. Solid Signs parameters.</p>			Parameter	Value	Ground to Top of Wall/Sign, h	18.394 ft	Wall/Sign Horizontal Dimension, B	73.500 ft	Wall/Sign Vertical Dimension, s	12.319 ft	Ratio of Solid Area to Gross Area, ϵ	1.000	Length of return corner, L_r	- ft
Parameter	Value													
Ground to Top of Wall/Sign, h	18.394 ft													
Wall/Sign Horizontal Dimension, B	73.500 ft													
Wall/Sign Vertical Dimension, s	12.319 ft													
Ratio of Solid Area to Gross Area, ϵ	1.000													
Length of return corner, L_r	- ft													

Figure 26.5-1	<p>Basic Wind Speed, V</p> <p>Wind speed for the address is 105 mph (defined by the user) for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.</p>	$V = 105$ mph (defined by the user)																																																																																
Figure 26.8-1	<p>Topographic Effects, K_{zt}</p> <p>The topography factor, K_{zt}, have been calculated based on the wind coming from N. K_{zt} was calculated using the following formulas:</p> $K_{zt} = (1 + K_1 K_2 K_3)^2$ $K_2 = (1 - x /\mu L_h)$ $K_3 = e^{-z/L_h}$ <p>and K_1 - determined from Figure 26.8-1</p> <p>Since the topography is classified as Escarpment, topography effects should be considered. From Section 26.8.1, K_{zt} is calculated (greater than 1.0) if the location satisfies all of the following conditions:</p> <p>If $H/L_h > 0.2$</p> $H/L_h = 8.154841545685485/45.370 = 0.180$ <p>Since $H < 15$ ft and Exposure category is C, $K_{zt} = 1.0$.</p>	$K_{zt} = 1.0$																																																																																
Table 26.6-1	<p>Wind Directionality Factor, K_d</p> <p>The wind directionality factors, K_d, for the structure is equal to 0.85 (for MWFRS, and Components and Claddings) based on Table 26.6-1.</p>	$K_d = 0.85$																																																																																
Section 26.9.1	<p>Gust Effect Factor, G</p> <p>The structure is assumed to be rigid, hence, gust effect factor, G, is set to 0.85 based on Section 26.9.1.</p>	$G = 0.85$																																																																																
Table 26.9-1	<p>Groud Elevation Factor, K_e</p> <p>The location is elevated at 6303.24 ft above mean sea level. To account for air density, K_e is calculated in accordance with Table 26.9-1 using the formula:</p> $K_e = e^{-0.0000362z_p}$ $K_e = e^{-0.0000362(6303.24)} = 0.796$	$K_e = 0.796$																																																																																
Section 26.10 Table 26.10-1	<p>Velocity Pressure Exposure Coefficient, K_z and Velocity Pressure, q_z</p> <p>The velocity pressures, q_z, shall be computed using the equation:</p> $q_z = 0.00256 K_z K_{zt} K_d K_e V^2$ $q_z = 0.00256 K_z (1)(0.85)(0.796)(105)^2$ <p>where: K_z is calculated for each height using Table 27.3-1 rounded to nearest hundredth. The table below shows the comparison of calculated q_z values for each parameter depending on the Exposure Category of each wind source direction to generate the worst case wind direction:</p> <table border="1" data-bbox="384 1272 1211 1496"> <thead> <tr> <th>Wind Direction</th> <th>Exposure Category</th> <th>Velocity Pressure Exposure Coefficient K_z @ 18.394 ft</th> <th>Topographic factor K_{zt} @ $z = 0$ ft</th> <th>Wind Directionality factor K_d</th> <th>Ground Elevation factor K_e</th> <th>Basic Wind Speed V, mph</th> <th>Velocity Pressure q_h, psf</th> </tr> </thead> <tbody> <tr> <td>N</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>S</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>E</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>W</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>NE</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>SE</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>NW</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> <tr> <td>SW</td> <td>C</td> <td>0.890</td> <td>1.000</td> <td>0.850</td> <td>0.796</td> <td>105.000</td> <td>16.995</td> </tr> </tbody> </table> <p>From the formula above, the calculated K_z and q_z per level for Wind Source Direction N - Exposure Category C are as follows:</p> <table border="1" data-bbox="550 1559 1043 1630"> <thead> <tr> <th>Level</th> <th>Height, ft</th> <th>K_z</th> <th>q_z, psf</th> </tr> </thead> <tbody> <tr> <td>Ground to Top of Wall/Sign</td> <td>18.394</td> <td>0.89</td> <td>17.00</td> </tr> </tbody> </table>	Wind Direction	Exposure Category	Velocity Pressure Exposure Coefficient K_z @ 18.394 ft	Topographic factor K_{zt} @ $z = 0$ ft	Wind Directionality factor K_d	Ground Elevation factor K_e	Basic Wind Speed V , mph	Velocity Pressure q_h , psf	N	C	0.890	1.000	0.850	0.796	105.000	16.995	S	C	0.890	1.000	0.850	0.796	105.000	16.995	E	C	0.890	1.000	0.850	0.796	105.000	16.995	W	C	0.890	1.000	0.850	0.796	105.000	16.995	NE	C	0.890	1.000	0.850	0.796	105.000	16.995	SE	C	0.890	1.000	0.850	0.796	105.000	16.995	NW	C	0.890	1.000	0.850	0.796	105.000	16.995	SW	C	0.890	1.000	0.850	0.796	105.000	16.995	Level	Height, ft	K_z	q_z , psf	Ground to Top of Wall/Sign	18.394	0.89	17.00	
Wind Direction	Exposure Category	Velocity Pressure Exposure Coefficient K_z @ 18.394 ft	Topographic factor K_{zt} @ $z = 0$ ft	Wind Directionality factor K_d	Ground Elevation factor K_e	Basic Wind Speed V , mph	Velocity Pressure q_h , psf																																																																											
N	C	0.890	1.000	0.850	0.796	105.000	16.995																																																																											
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Ground to Top of Wall/Sign	18.394	0.89	17.00																																																																															
Figure 29.3-1 of ASCE 7-16	<p>Net Force Coefficient, C_f</p> <p>The net force coefficients, C_f, for Case A and Case B are calculated using Figure 29.3-1 of ASCE 7-16. Note that the values are interpolated using known values for each s/h and B/s value:</p> $B/s = 73.50/12.32 = 5.966$ $s/h = 12.32/18.39 = 0.670$ <p>Reduction Factor for signs with opening:</p> $R_{factor,open} = 1 - (1 - \epsilon)^{1.5} = 1 - (1 - 1.000)^{1.5} = 1.000$ <p>For Case A:</p> $C_{f,A} = R_{factor,open} C_{f,A} = (1.000)(1.573) = 1.573$ <p>For Case B:</p> $C_{f,B} = R_{factor,open} C_{f,B} = (1.000)(1.573) = 1.573$ <p>For Case C:</p>																																																																																	

reduction factor for return corners:

$$R_{factor,Lf/s} = 1.000$$

Distance from windward edge, ft	$R_{factor,open}$	$R_{factor,s/h}$	Initial C_f	Final $C_{f,C}$
0 to s	1.000	1.000	3.293	3.293
s to 2s	1.000	1.000	2.145	2.145
2s to 3s	1.000	1.000	1.547	1.547
3s to 10s	1.000	1.000	1.050	1.050

Equation 29.3-1 of ASCE 7-16

Design wind Force, F

The design wind force, F , can be calculated using Equation 29.3-1 of ASCE 7-16.

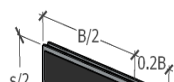
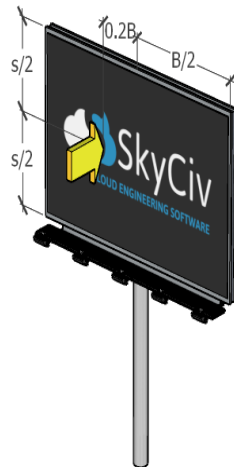
$$F = q_h G C_f A_s = (17.00)(0.85)C_f(905.45) = 13080.161C_f$$

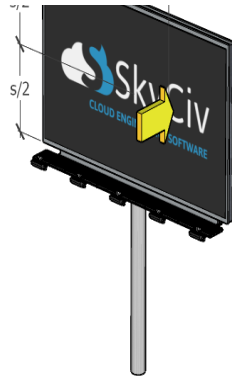
The design forces for each case is summarized on table below:

Case	Location	C_f	Design Force, F lb
Case A	e = 0 ft	1.573	20571.209
Case B	e = 14.70 ft	1.573	20571.209
Case C	0 to s	3.293	7219.87
	s to 2s	2.145	4702.41
	2s to 3s	1.547	3390.71
	3s to 10s	1.050	6828.40



Figure 3. Case A.



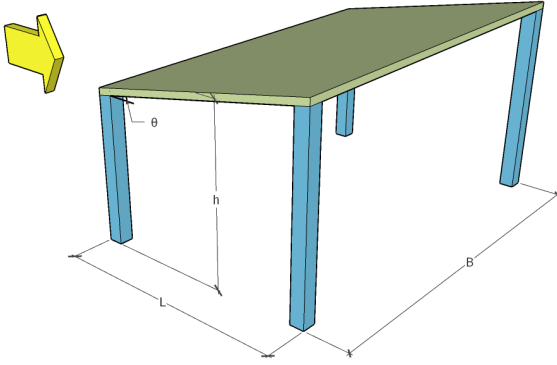




Figures 4 and 5. Case B.



Figures 6. Case C.

REFERENCES	CALCULATIONS	RESULTS												
	<p style="text-align: center;">Snow Load Detailed Calculations based on ASCE 7-16</p> <p>Design Information :</p> <p>Project Name : Beehive11a-JB-RevA Client : Designer : MT_SKYCIV AutoDesigner Company : MT Solar Units : Imperial Notes : Snow loads based on monoslope structure</p>  <p>Project Data</p> <p>The structure is located in Big Sky, MT, USA categorized as Risk Category I. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at 6303 ft above mean sea level.</p>  <p style="text-align: center;">Figure 1. Site location.</p> <p>Additional details of the structure are shown in Table below and illustrated in Figure 2:</p> <table border="1" data-bbox="590 1182 1002 1415"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Building Length, L</td> <td>11.896 ft</td> </tr> <tr> <td>Building Width, B</td> <td>74.250 ft</td> </tr> <tr> <td>Mean Roof Height, h</td> <td>12.234 ft</td> </tr> <tr> <td>Roof Profile</td> <td>Open Monoslope</td> </tr> <tr> <td>Roof Pitch Angle, θ</td> <td>46.000°</td> </tr> </tbody> </table>  <p style="text-align: center;">Figure 2. Building parameters.</p>	Parameter	Value	Building Length, L	11.896 ft	Building Width, B	74.250 ft	Mean Roof Height, h	12.234 ft	Roof Profile	Open Monoslope	Roof Pitch Angle, θ	46.000°	
Parameter	Value													
Building Length, L	11.896 ft													
Building Width, B	74.250 ft													
Mean Roof Height, h	12.234 ft													
Roof Profile	Open Monoslope													
Roof Pitch Angle, θ	46.000°													
<p>Section 7.2 of ASCE 7</p>	<p>Ground Snow Load, p_g</p> <p>The ground snow load, p_g, for the site location is 80 psf (defined by the user) at elevation 6303.24 ft above mean sea level based on Section 7.2 of ASCE 7.</p>	<p>$p_g = 80$ psf (defined by the user)</p>												

	Based on Section 7.2 of ASCE 7.	by the user
Table 7-2 Section 7.3.1 of ASCE 7	<p>Exposure Factor, C_e</p> <p>The exposure factor, C_e, for the structure is equal 0.90 as the terrain is categorized as Exposure C with exposure condition specified as Fully Exposed based on Table 7-2 Section 7.3.1 of ASCE 7.</p>	$C_e = 0.90$
Table 7-3 Section 7.3.2 of ASCE 7	<p>Thermal Factor, C_t</p> <p>Since the thermal condition of the structure is categorized as "Unheated and open air structures," the corresponding thermal factor, C_t, is equal 1.20 based on Table 7-3 Section 7.3.2 of ASCE 7.</p>	$C_t = 1.20$
Table 1.5-2 of Chapter 1 ASCE 7	<p>Importance Factor, I_s</p> <p>Since the structure is classified Risk Category I, the Importance Factor, I_s, is equal to 0.8.</p>	$I_s = 0.80$
Equation 7.3-1 of Section 7.3 ASCE 7	<p>Flat Roof Snow Load, p_f</p> <p>The flat roof snow load, p_f, (psf) is calculated using the Equation 7.3-1:</p> $p_f = 0.7C_eC_tI_s p_g$ $p_f = 0.7(0.90)(1.20)(0.80)(80.00) = 48.38psf$	$p_f = 48.38$ psf
Section 7.10 ASCE 7	<p>Rain-on-snow Surcharge Load, p_r</p> <p>The rain-on-snow surcharge load, p_r, is equal to 0.00 psf since $p_g > 20$ psf.</p>	$p_r = 0.00$ psf
Equation 7.7-1 of ASCE 7	<p>Snow Density, γ</p> <p>The snow density, γ, is calculated using Equation 7.7-1 of ASCE 7 as:</p> $\gamma = 0.13p_g + 14 \leq 30 = 0.13(80.00) + 14 \leq 30$ $\gamma = 24.40pcf$	$\gamma = 24.40pcf$
Section 7.4 ASCE 7	<p>Roof Slope Factor (Balanced), C_s</p> <p>Since the roof is classified as cold roof ($C_t > 1.0$), the corresponding roof slope factor, C_s, is equal to 0.436 based on Figure 7.2c where $\theta = 46.00^\circ$.</p>	$C_s = 0.436$
Equation 7.4-1 of Section 7.4 ASCE 7	<p>Sloped Roof Snow Load (Balanced), p_s</p> <p>The sloped roof snow load, p_s, (psf) is calculated using the Equation 7.4-1:</p> $p_s = C_s p_f$ $p_s = (0.436)(48.38) = 21.11psf$	$p_s = 21.11$ psf

Project Details

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

User Name: sales@mtsolar.us
 Project Name: Beehive11a-JB-RevA
 Unit System: imperial



Design Input Information

Design Factors			
Φ_t	Φ_c	Φ_b	Φ_v
0.9	0.9	0.9	0.9

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

Section Dimensions

ID	Name	d (in)	t _w (in)				
2	2in Pipe Sch 80	2.38	0.22				
5	4in Pipe Sch 80	4.50	0.34				
11	10in Pipe Sch 40	10.75	0.36				

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

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

Section Properties								
ID	Name	A (in ²)	J (in ⁴)	I _{yp} (in ⁴)	I _{zp} (in ⁴)	I _w (in ⁶)	S _{yp} (in ³)	S _{zp} (in ³)
2	2in Pipe Sch 80	1.48	1.74	0.87	0.87	0.00	1.02	1.02
5	4in Pipe Sch 80	4.41	19.22	9.61	9.61	0.00	5.85	5.85

11	10in Pipe Sch 40	11.91	321.47	160.73	160.73	0.00	39.38	39.38
16	HSS5x3x3/16	2.58	8.64	3.85	8.53	92.39	2.96	4.21
19	W8x10	2.96	0.04	2.09	30.80	30.90	1.66	8.87

Member Properties														
Member ID	Section ID	K _z L (ft)	K _y L (ft)	L _b (ft)	C _b	L	S	T	L	S	T	L	S	T
1	11	25.69	25.69	12.23	-	3	0	0	2	0	0	1		
2	5	1.30	1.30	2.00	-	3	0	0	2	0	0	1		
3	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.16,1.19,1.18,1.19,1.17,1.19	3	0	0	2	0	0	1		
4	16	2.44	2.44	3.75	1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.68,1.65,1.68,1.67,1.69,1.66,1.69,1.67,1.67,1.68,1.67,1.67,1.69,1.64,1.69,1.67,1.69,1.66,1.69	3	0	0	2	0	0	1		
5	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		
6	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	3	0	0	2	0	0	1		
7	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		
8	19	1.33	1.33	2.05	1.74,1.75,1.74,1.75,1.74,1.74,1.74,1.75,1.74,1.75,1.74,1.74,1.74,1.74,1.74,1.74,1.75,1.75,1.75,1.74,1.74,1.74,1.74,1.74,1.74,1.74	3	0	0	2	0	0	1		
9	2	2.60	2.60	4.00	-	3	0	0	2	0	0	1		
10	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		
11	19	1.33	1.33	2.05	1.77,1.77,1.77,1.78,1.77,1.77,1.84,1.77,2.04,1.77,1.86,1.77,1.92,1.77,1.81,1.78,1.71,1.78,1.84,1.77,2.03,1.77,1.86,1.77,1.90,1.77	3	0	0	2	0	0	1		
12	5	1.30	1.30	2.00	-	3	0	0	2	0	0	1		
13	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.08,1.10,1.09,1.10,1.09,1.10,1.10,1.10,1.11,1.10,1.10,1.10,1.08,1.10,1.09,1.10,1.09,1.10,1.09,1.10	3	0	0	2	0	0	1		
14	19	4.88	4.00	7.50	1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.10,1.10,1.10,1.10,1.09,1.10,1.10,1.10,1.10,1.10	3	0	0	2	0	0	1		
15	19	7.88	7.88	3.75	2.33,2.33	3	0	0	2	0	0	1		
16	19	7.88	7.88	3.75	2.33,2.33	3	0	0	2	0	0	1		
101	11	25.69	25.69	12.23	-	3	0	0	2	0	0	1		
102	5	1.30	1.30	2.00	-	3	0	0	2	0	0	1		
103	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.17,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	3	0	0	2	0	0	1		
104	16	2.44	2.44	3.75	1.68,1.68,1.68,1.67,1.68,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		
105	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		
106	16	0.92	0.92	1.42	1.19,1.18,1.19,1.18,1.18,1.19,1.18,1.18,1.17,1.18,1.18,1.19,1.18,1.19,1.18,1.18,1.18,1.18,1.18,1.19,1.17,1.19,1.18,1.19,1.18,1.19	3	0	0	2	0	0	1		
107	16	1.52	1.52	2.33	1.68,1.67,1.68,1.67,1.67,1.68,1.67,1.67,1.66,1.67,1.67,1.68,1.66,1.68,1.67,1.67,1.67,1.67,1.67,1.68,1.65,1.68,1.67,1.68,1.66,1.68	3	0	0	2	0	0	1		

103	116.10	115.41	15.79	11.10	42.08	23.28
104	116.10	111.33	15.79	11.10	42.08	23.28
105	116.10	114.23	15.79	11.10	42.08	23.28
106	116.10	115.41	15.79	11.10	42.08	23.28
107	116.10	114.23	15.79	11.10	42.08	23.28
108	133.20	126.01	32.87	6.12	40.24	43.62
109	66.48	58.89	3.82	3.82	19.94	19.94
110	116.10	111.33	15.79	11.10	42.08	23.28
111	133.20	126.01	32.87	6.12	40.24	43.62
112	198.33	196.72	21.95	21.95	59.50	59.50
113	133.20	104.94	23.83	6.12	40.24	43.62
114	133.20	104.94	23.83	6.12	40.24	43.62
115	133.20	69.16	17.33	6.12	40.24	43.62
116	133.20	69.16	17.64	6.12	40.24	43.62
201	535.87	320.22	147.68	147.68	160.76	160.76
202	198.33	196.72	21.95	21.95	59.50	59.50
203	116.10	115.41	15.79	11.10	42.08	23.28
204	116.10	111.33	15.79	11.10	42.08	23.28
205	116.10	114.23	15.79	11.10	42.08	23.28
206	116.10	115.41	15.79	11.10	42.08	23.28
207	116.10	114.23	15.79	11.10	42.08	23.28
208	133.20	126.01	32.87	6.12	40.24	43.62
209	66.48	58.89	3.82	3.82	19.94	19.94
210	116.10	111.33	15.79	11.10	42.08	23.28
211	133.20	126.01	32.87	6.12	40.24	43.62
212	198.33	196.72	21.95	21.95	59.50	59.50
213	133.20	104.94	23.83	6.12	40.24	43.62
214	133.20	104.94	23.83	6.12	40.24	43.62
215	133.20	69.16	17.49	6.12	40.24	43.62
216	133.20	69.16	17.80	6.12	40.24	43.62
301	535.87	320.22	147.68	147.68	160.76	160.76
302	198.33	196.72	21.95	21.95	59.50	59.50
303	116.10	115.41	15.79	11.10	42.08	23.28
304	116.10	111.33	15.79	11.10	42.08	23.28
305	116.10	114.23	15.79	11.10	42.08	23.28
306	116.10	115.41	15.79	11.10	42.08	23.28
307	116.10	114.23	15.79	11.10	42.08	23.28
308	133.20	52.83	32.87	6.12	40.24	43.62
309	66.48	58.89	3.82	3.82	19.94	19.94
310	116.10	111.33	15.79	11.10	42.08	23.28
311	133.20	52.83	32.87	6.12	40.24	43.62
312	198.33	196.72	21.95	21.95	59.50	59.50
313	133.20	104.94	24.75	6.12	40.24	43.62
314	133.20	104.94	24.98	6.12	40.24	43.62
315	133.20	69.16	17.33	6.12	40.24	43.62
316	133.20	69.16	17.33	6.12	40.24	43.62

Design Ratio

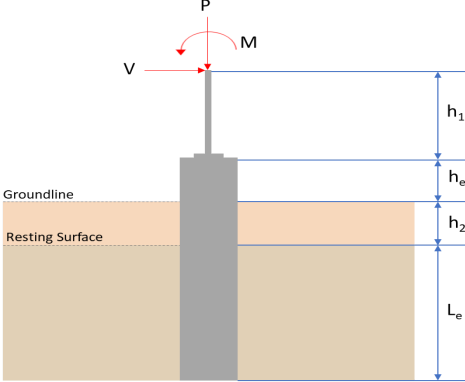
Member ID	P	M _z	M _y	V _y	V _z	(P,M _z ,M _y)	Worst LC	KL/r	δ	Status
1	0.036	0.312	0.013	0.023	0.001	0.329	#13	0.420	Not Required	Pass
2	0.004	0.342	0.164	0.084	0.030	0.470	#21	0.035	Not Required	Pass
3	0.013	0.498	0.062	0.049	0.005	0.565	#21	0.045	Not Required	Pass

4	0.012	0.493	0.198	0.050	0.043	0.637	#21	0.080	Not Required	Pass
5	0.012	0.309	0.196	0.050	0.049	0.359	#21	0.074	Not Required	Pass
6	0.016	0.568	0.116	0.057	0.023	0.692	#21	0.045	Not Required	Pass
7	0.017	0.353	0.268	0.056	0.067	0.423	#21	0.074	Not Required	Pass
8	0.002	0.067	0.233	0.038	0.026	0.239	#23	0.095	Not Required	Pass
9	0.018	0.038	0.076	0.002	0.002	0.105	#21	0.204	Not Required	Pass
10	0.017	0.558	0.254	0.056	0.054	0.729	#21	0.080	Not Required	Pass
11	0.003	0.065	0.240	0.038	0.026	0.249	#21	0.095	Not Required	Pass
12	0.004	0.427	0.192	0.103	0.035	0.569	#21	0.035	Not Required	Pass
13	0.009	0.173	0.618	0.049	0.034	0.753	#21	0.286	Not Required	Pass
14	0.009	0.174	0.610	0.049	0.034	0.737	#21	0.190	Not Required	Pass
15	0.000	0.055	0.216	0.024	0.016	0.271	#21	Not Required	Not Required	Pass
16	0.000	0.055	0.216	0.024	0.016	0.271	#21	Not Required	Not Required	Pass
101	0.041	0.359	0.001	0.027	0.000	0.374	#13	0.420	Not Required	Pass
102	0.005	0.441	0.203	0.107	0.035	0.598	#21	0.035	Not Required	Pass
103	0.016	0.613	0.099	0.061	0.015	0.720	#21	0.045	Not Required	Pass
104	0.016	0.610	0.258	0.061	0.055	0.791	#21	0.080	Not Required	Pass
105	0.016	0.381	0.269	0.061	0.068	0.451	#21	0.074	Not Required	Pass
106	0.016	0.615	0.099	0.061	0.015	0.721	#21	0.045	Not Required	Pass
107	0.016	0.382	0.266	0.061	0.067	0.452	#21	0.074	Not Required	Pass
108	0.002	0.048	0.231	0.039	0.026	0.266	#21	0.095	Not Required	Pass
109	0.021	0.037	0.058	0.001	0.000	0.102	#21	0.204	Not Required	Pass
110	0.016	0.609	0.255	0.061	0.054	0.789	#21	0.080	Not Required	Pass
111	0.003	0.051	0.237	0.039	0.026	0.270	#21	0.095	Not Required	Pass
112	0.005	0.441	0.207	0.107	0.036	0.600	#21	0.035	Not Required	Pass
113	0.009	0.191	0.621	0.051	0.034	0.791	#21	0.286	Not Required	Pass
114	0.010	0.199	0.615	0.051	0.034	0.791	#21	0.286	Not Required	Pass
115	0.006	0.233	0.343	0.040	0.027	0.579	#21	0.473	Not Required	Pass
116	0.002	0.233	0.342	0.040	0.027	0.576	#21	0.473	Not Required	Pass
201	0.041	0.359	0.001	0.027	0.000	0.374	#13	0.420	Not Required	Pass
202	0.005	0.441	0.207	0.107	0.036	0.600	#21	0.035	Not Required	Pass
203	0.016	0.615	0.099	0.061	0.015	0.721	#21	0.045	Not Required	Pass
204	0.016	0.609	0.255	0.061	0.054	0.789	#21	0.080	Not Required	Pass
205	0.016	0.382	0.266	0.061	0.067	0.452	#21	0.074	Not Required	Pass
206	0.016	0.613	0.099	0.061	0.015	0.720	#21	0.045	Not Required	Pass
207	0.016	0.381	0.269	0.061	0.068	0.451	#21	0.074	Not Required	Pass
208	0.002	0.055	0.237	0.040	0.027	0.267	#21	0.095	Not Required	Pass
209	0.021	0.037	0.058	0.001	0.000	0.102	#21	0.204	Not Required	Pass
210	0.016	0.610	0.258	0.061	0.055	0.791	#21	0.080	Not Required	Pass
211	0.003	0.058	0.243	0.040	0.027	0.270	#21	0.095	Not Required	Pass
212	0.005	0.441	0.203	0.107	0.035	0.598	#21	0.035	Not Required	Pass
213	0.009	0.191	0.621	0.051	0.034	0.791	#21	0.286	Not Required	Pass
214	0.010	0.199	0.615	0.051	0.034	0.791	#21	0.286	Not Required	Pass
215	0.006	0.211	0.343	0.039	0.026	0.558	#21	0.473	Not Required	Pass
216	0.002	0.206	0.342	0.039	0.026	0.549	#21	0.473	Not Required	Pass
301	0.036	0.312	0.013	0.023	0.001	0.329	#13	0.420	Not Required	Pass
302	0.004	0.427	0.192	0.103	0.035	0.569	#21	0.035	Not Required	Pass
303	0.016	0.568	0.116	0.057	0.023	0.692	#21	0.045	Not Required	Pass
304	0.017	0.558	0.254	0.056	0.054	0.729	#21	0.080	Not Required	Pass
305	0.017	0.353	0.268	0.056	0.067	0.423	#21	0.074	Not Required	Pass
306	0.013	0.498	0.062	0.049	0.005	0.565	#21	0.045	Not Required	Pass
307	0.012	0.309	0.196	0.050	0.049	0.359	#21	0.074	Not Required	Pass
308	0.000	0.055	0.216	0.024	0.016	0.271	#21	Not Required	Not Required	Pass
309	0.018	0.038	0.076	0.002	0.002	0.105	#21	0.204	Not Required	Pass

309	0.018	0.058	0.078	0.002	0.002	0.105	#21	0.204	Not Required	Pass
310	0.012	0.493	0.198	0.050	0.043	0.637	#21	0.080	Not Required	Pass
311	0.000	0.055	0.216	0.024	0.016	0.271	#21	Not Required	Not Required	Pass
312	0.004	0.342	0.164	0.084	0.030	0.470	#21	0.035	Not Required	Pass
313	0.009	0.173	0.618	0.049	0.034	0.753	#21	0.190	Not Required	Pass
314	0.009	0.174	0.610	0.049	0.034	0.737	#21	0.286	Not Required	Pass
315	0.006	0.236	0.343	0.038	0.026	0.582	#21	0.473	Not Required	Pass
316	0.002	0.238	0.342	0.038	0.026	0.580	#21	0.473	Not Required	Pass

Definitions

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

REFERENCES	CALCULATIONS	RESULTS																										
	<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: round $D = 96 \text{ in}$ - Pile diameter $L = 6.25 \text{ ft}$ - Total pile length $h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile, $h_2 = 0 \text{ ft}$ - Depth to resisting surface $h_e = 0 \text{ ft}$ - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="517 1079 1094 1169"> <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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1263 935 1438"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.284</td> <td>11.674</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.239</td> <td>-3.739</td> </tr> <tr> <td>V_z (kip)</td> <td>0.129</td> <td>0.208</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.420</td> <td>0.687</td> </tr> <tr> <td>M_z (kipft)</td> <td>27.457</td> <td>46.051</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength.</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	7.284	11.674	V_x (kip)	-2.239	-3.739	V_z (kip)	0.129	0.208	M_x (kipft)	0.420	0.687	M_z (kipft)	27.457	46.051	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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M_z (kipft)	27.457	46.051																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.239 \text{ kip})}{(96 \text{ in})}$ $H_o = -0.27987 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(27.457 \text{ kipft}) + ((-2.239 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 3.4321 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9109 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.129 \text{ kip})}{(96 \text{ in})}$$

$$H_o = 0.016125 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.42 \text{ kipft}) + ((0.129 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.0525 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.926 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9109 \text{ ft}), (1.926 \text{ ft})]$$

$$L_{e,req} = 5.911 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.911 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94576$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(96 \text{ in})}{2}\right)^2$$

$$A = 50.265 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.284 \text{ kip})}{(50.265 \text{ ft}^2)}$$

$$q = 0.14491 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.14491 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.048304$$

Status: **PASS**
Ratio: **0.050**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(96 \text{ in})}$$

$$L/D = 0.78125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.27987 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.4321 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4321 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.4321 \text{ kipft/ft})) + (4 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2988 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.4321 \text{ kipft/ft})) + (3 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.4321 \text{ kipft/ft})) + (2 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.31907 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.4321 \text{ kipft/ft})) + ((-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 1.2342 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.2988 \text{ ft})}{2}$$

$$p_a = 0.42988 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.31907 \text{ kip/ft}^2)}{(0.42988 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74224$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 1.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2342 \text{ kip/ft}^2)}{(1.25 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98732$$

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = 0.016125 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0525 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0525 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.016125 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0525 \text{ kipft/ft})) + (4 \times (0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.459 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.0525 \text{ kipft/ft})) + (3 \times (0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0525 \text{ kipft/ft})) + (2 \times (0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.022046 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.0525 \text{ kipft/ft})) + ((0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.049651 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.459 \text{ ft})}{2}$$

$$p_a = 0.4459 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.022046 \text{ kip/ft}^2)}{(0.4459 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.049442$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 1.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

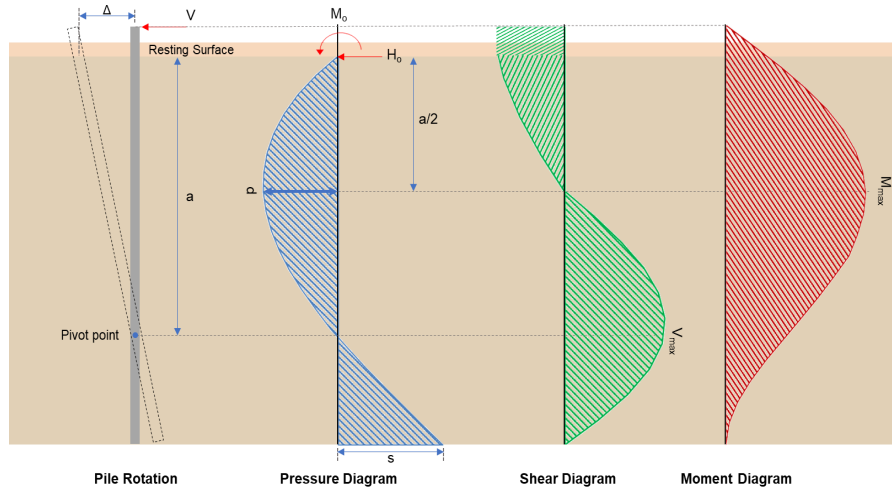
Status: **PASS**
Ratio: **0.050**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.049651 \text{ kip/ft}^2)}{(1.25 \text{ kip/ft}^2)}$$

$$Ratio = 0.039721$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.739 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.46737 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.051 \text{ kipft}) + ((-3.739 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 5.7564 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.7564 \text{ kipft/ft})}{(-0.46737 \text{ kip/ft})}$$

$$E = 12.316 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.7564 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.46737 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.7564 \text{ kipft/ft})) + (4 \times (-0.46737 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2983 \text{ ft}$$

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

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

$$V_{max} = ((-0.46737 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.316 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2983 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (12.316 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2983 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.506 \text{ kip}$$

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

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

$$M_{max} = ((-0.46737 \text{ kip/ft}) \times (96 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(12.316 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.316 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.316 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 46.332 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.208 \text{ kip})}{(96 \text{ in})}$$

$$H_o = 0.026 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.687 \text{ kipft}) + ((0.208 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.085875 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.085875 \text{ kipft/ft})}{(0.026 \text{ kip/ft})}$$

$$E = 3.3029 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.085875 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.026 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.085875 \text{ kipft/ft})) + (4 \times (0.026 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4572 \text{ ft}$$

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

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

$$V_{max} = ((0.026 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3029 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4572 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.3029 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4572 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33297 \text{ kip}$$

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

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

$$M_{max} = ((0.026 \text{ kip/ft}) \times (96 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.3029 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4572 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.3029 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4572 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3029 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4572 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.9245 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 7238.2 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.674 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (7238.2 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (7238.2 \text{ in}^2)) \right]$$

$$A_{st,required} = -265.4 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-265.4 \text{ in}^2), (0.0018 \times (7238.2 \text{ in}^2))]$$

$$A_{min} = 13.029 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(13.029 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 44$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (44) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 13.499 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.029 \text{ in}^2)}{(13.499 \text{ in}^2)}$$

$$\text{Ratio} = 0.96517$$

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 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (96 \text{ in})]$$

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

Summary:

Main reinforcement: **44 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(7238.2 \text{ in}^2) - (13.499 \text{ in}^2)]) + ((60 \text{ ksi}) \times (13.499 \text{ in}^2))]$$

$$\phi P_N = 8929.8 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.674 \text{ kip})}{(8929.8 \text{ kip})}$$

$$\text{Ratio} = 0.0013073$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

22.5.2.2

$b_w = 96 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (96 \text{ in})$$

$$d = 76.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(76.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.48002$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.48002) \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,max} = 884.76 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.674 \text{ kip} \rightarrow 11674 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + \frac{(11674 \text{ lbf})}{6 \times (7238.2 \text{ in}^2)} \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,a} = 355.89 \text{ kip}$$

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

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,b} = 1275.5 \text{ kip}$$

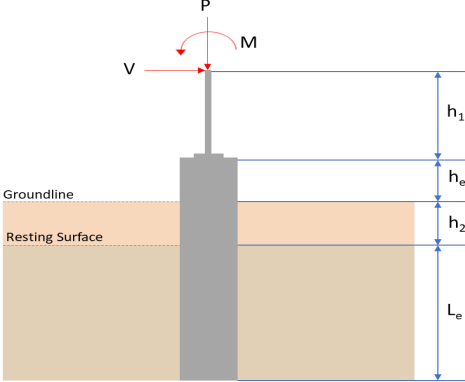
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(884.76 \text{ kip}), (355.89 \text{ kip}), (1275.5 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 355.89 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$ $V_{s,a} = 2949.1 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (76.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 101.79 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(2949.1 \text{ kip}), (101.79 \text{ kip})]$ $V_s = 101.79 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((355.89 \text{ kip}) + (101.79 \text{ kip}))$ $\phi V_n = 297.49 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 15.506 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(15.506 \text{ kip})}{(297.49 \text{ kip})}$ $Ratio = 0.052124$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33297 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.33297 \text{ kip})}{(297.49 \text{ kip})}$ $Ratio = 0.0011193$	<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> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (96 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 86859 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 86858.753 \text{ in}^3$ $\phi M_{n,1} = 1176.212 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (86859 \text{ in}^3)$ $\phi M_{n,2} = 9997.8 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(1176.2 \text{ kipft}), (9997.8 \text{ kipft})]$ $\phi M_n = 1176.2 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 46.332 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(46.332 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.03939$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 0.9245 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.9245 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.00078599$	<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: round $D = 96 \text{ in}$ - Pile diameter $L = 6.25 \text{ ft}$ - Total pile length $h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile, $h_2 = 0 \text{ ft}$ - Depth to resisting surface $h_e = 0 \text{ ft}$ - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="515 1079 1094 1171"> <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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.284</td> <td>11.674</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.239</td> <td>-3.739</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.129</td> <td>-0.208</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.420</td> <td>-0.688</td> </tr> <tr> <td>M_z (kipft)</td> <td>27.457</td> <td>46.052</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength.</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	7.284	11.674	V_x (kip)	-2.239	-3.739	V_z (kip)	-0.129	-0.208	M_x (kipft)	-0.420	-0.688	M_z (kipft)	27.457	46.052	
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M_z (kipft)	27.457	46.052																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.239 \text{ kip})}{(96 \text{ in})}$ $H_o = -0.27987 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(27.457 \text{ kipft}) + ((-2.239 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 3.4321 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9109 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.129 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.016125 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.42 \text{ kipft}) + ((-0.129 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.0525 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.4824 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9109 \text{ ft}), (1.4824 \text{ ft})]$$

$$L_{e,req} = 5.911 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.911 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94576$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(96 \text{ in})}{2}\right)^2$$

$$A = 50.265 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(7.284 \text{ kip})}{(50.265 \text{ ft}^2)}$$

$$q = 0.14491 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.14491 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.048304$$

Status: **PASS**
Ratio: **0.050**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(96 \text{ in})}$$

$$L/D = 0.78125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.27987 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.4321 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.4321 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.4321 \text{ kipft/ft})) + (4 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2988 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.4321 \text{ kipft/ft})) + (3 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.4321 \text{ kipft/ft})) + (2 \times (-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.31907 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.4321 \text{ kipft/ft})) + ((-0.27987 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 1.2342 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.2988 \text{ ft})}{2}$$

$$p_a = 0.42988 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.31907 \text{ kip/ft}^2)}{(0.42988 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74224$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 1.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2342 \text{ kip/ft}^2)}{(1.25 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98732$$

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = -0.016125 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.0525 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.0525 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.016125 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.0525 \text{ kipft/ft})) + (4 \times (-0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.459 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.0525 \text{ kipft/ft})) + (3 \times (-0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.0525 \text{ kipft/ft})) + (2 \times (-0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.0058362 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.0525 \text{ kipft/ft})) + ((-0.016125 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.0010179 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.459 \text{ ft})}{2}$$

$$p_a = 0.4459 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.0058362 \text{ kip/ft}^2)}{(0.4459 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.013088$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.25 \text{ ft})$$

$$p_s = 1.25 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

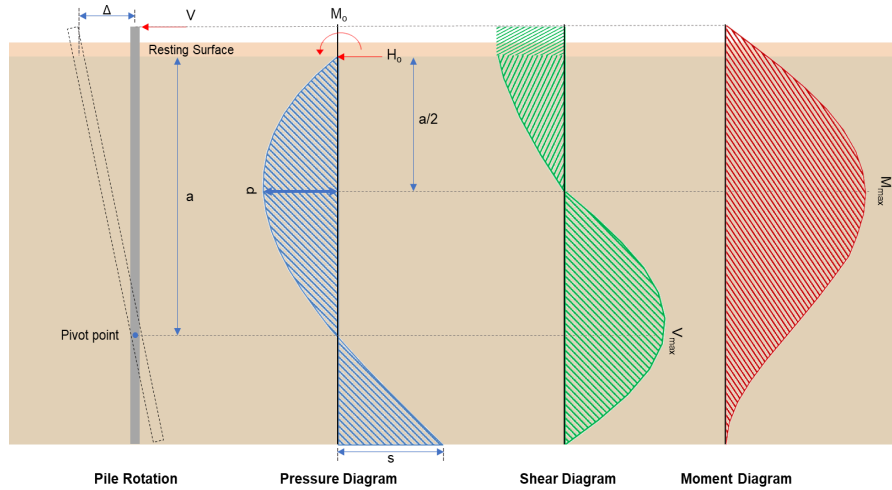
Status: **PASS**
Ratio: **-0.010**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0010179 \text{ kip/ft}^2)}{(1.25 \text{ kip/ft}^2)}$$

$$Ratio = 0.00081432$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-3.739 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.46737 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(46.052 \text{ kipft}) + ((-3.739 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 5.7565 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.7565 \text{ kipft/ft})}{(-0.46737 \text{ kip/ft})}$$

$$E = 12.317 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.7565 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.46737 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (5.7565 \text{ kipft/ft})) + (4 \times (-0.46737 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2983 \text{ ft}$$

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

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

$$V_{max} = ((-0.46737 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.317 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2983 \text{ ft})}{(6.25 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (12.317 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2983 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 15.506 \text{ kip}$$

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

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

$$M_{max} = ((-0.46737 \text{ kip/ft}) \times (96 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(12.317 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.317 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.317 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2983 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 46.332 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.208 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.026 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.688 \text{ kipft}) + ((-0.208 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.086 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.086 \text{ kipft/ft})}{(-0.026 \text{ kip/ft})}$$

$$E = 3.3077 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.086 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.026 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.086 \text{ kipft/ft})) + (4 \times (-0.026 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.457 \text{ ft}$$

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

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

$$V_{max} = ((-0.026 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.3077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.457 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.3077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.457 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33325 \text{ kip}$$

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

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

$$M_{max} = ((-0.026 \text{ kip/ft}) \times (96 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(3.3077 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.457 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.3077 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.457 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.3077 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.457 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.92537 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 7238.2 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.674 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (7238.2 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (7238.2 \text{ in}^2)) \right]$$

$$A_{st,required} = -265.4 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-265.4 \text{ in}^2), (0.0018 \times (7238.2 \text{ in}^2))]$$

$$A_{min} = 13.029 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(13.029 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 44$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (44) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 13.499 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.029 \text{ in}^2)}{(13.499 \text{ in}^2)}$$

$$\text{Ratio} = 0.96517$$

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 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (96 \text{ in})]$$

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

Summary:

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

Main reinforcement: **44 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(7238.2 \text{ in}^2) - (13.499 \text{ in}^2)]) + ((60 \text{ ksi}) \times (13.499 \text{ in}^2))]$$

$$\phi P_N = 8929.8 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.674 \text{ kip})}{(8929.8 \text{ kip})}$$

$$\text{Ratio} = 0.0013073$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

$b_w = 96 \text{ in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (96 \text{ in})$$

$$d = 76.8 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(76.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.48002$$

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

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

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

$$V_{c,max} = 5 \times (0.48002) \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,max} = 884.76 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.674 \text{ kip} \rightarrow 11674 \text{ lbf}$.

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

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

$$V_{c,a} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + \frac{(11674 \text{ lbf})}{6 \times (7238.2 \text{ in}^2)} \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,a} = 355.89 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,b} = 1275.5 \text{ kip}$$

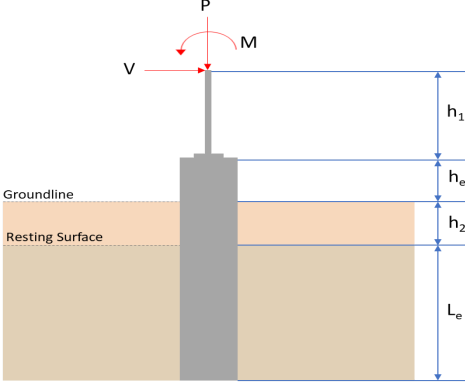
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(884.76 \text{ kip}), (355.89 \text{ kip}), (1275.5 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 355.89 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$ $V_{s,a} = 2949.1 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (76.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 101.79 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(2949.1 \text{ kip}), (101.79 \text{ kip})]$ $V_s = 101.79 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((355.89 \text{ kip}) + (101.79 \text{ kip}))$ $\phi V_n = 297.49 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 15.506 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(15.506 \text{ kip})}{(297.49 \text{ kip})}$ $Ratio = 0.052125$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33325 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.33325 \text{ kip})}{(297.49 \text{ kip})}$ $Ratio = 0.0011202$	<p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p>
	<p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (96 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 86859 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 86858.753 \text{ in}^3$ $\phi M_{n,1} = 1176.212 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (86859 \text{ in}^3)$ $\phi M_{n,2} = 9997.8 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(1176.2 \text{ kipft}), (9997.8 \text{ kipft})]$ $\phi M_n = 1176.2 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 46.332 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(46.332 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.039391$	<p>Status: PASS Ratio: 0.040</p>
	<p>Considering z-direction: $M_{max} = 0.92537 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.92537 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.00078674$	<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: round $D = 96 \text{ in}$ - Pile diameter $L = 6.5 \text{ ft}$ - Total pile length $h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile, $h_2 = 0 \text{ ft}$ - Depth to resting surface $h_e = 0 \text{ ft}$ - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="517 1079 1094 1169"> <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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1263 935 1438"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.266</td> <td>13.285</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.555</td> <td>-4.265</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.007</td> <td>-0.012</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.021</td> <td>-0.035</td> </tr> <tr> <td>M_z (kipft)</td> <td>31.427</td> <td>52.992</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	8.266	13.285	V_x (kip)	-2.555	-4.265	V_z (kip)	-0.007	-0.012	M_x (kipft)	-0.021	-0.035	M_z (kipft)	31.427	52.992	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
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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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.555 \text{ kip})}{(96 \text{ in})}$ $H_o = -0.31938 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(31.427 \text{ kipft}) + ((-2.555 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 3.9284 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.141 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.007 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.000875 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.021 \text{ kipft}) + ((-0.007 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.002625 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.59519 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.141 \text{ ft}), (0.59519 \text{ ft})]$$

$$L_{e,req} = 6.141 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.141 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.94477$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(96 \text{ in})}{2}\right)^2$$

$$A = 50.265 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.266 \text{ kip})}{(50.265 \text{ ft}^2)}$$

$$q = 0.16445 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.16445 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.054816$$

Status: **PASS**
Ratio: **0.050**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(96 \text{ in})}$$

$$L/D = 0.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.31938 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.9284 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9284 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.9284 \text{ kipft/ft})) + (4 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4744 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.9284 \text{ kipft/ft})) + (3 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.9284 \text{ kipft/ft})) + (2 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.32866 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.9284 \text{ kipft/ft})) + ((-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 1.2896 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.4744 \text{ ft})}{2}$$

$$p_a = 0.44744 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32866 \text{ kip/ft}^2)}{(0.44744 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73453$$

Status: **PASS**
Ratio: **0.730**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 1.3 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2896 \text{ kip/ft}^2)}{(1.3 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99197$$

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = -0.000875 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.002625 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.002625 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.000875 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.002625 \text{ kipft/ft})) + (4 \times (-0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6534 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.002625 \text{ kipft/ft})) + (3 \times (-0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.002625 \text{ kipft/ft})) + (2 \times (-0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = -0.00034308 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.002625 \text{ kipft/ft})) + ((-0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = -0.000097596 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.6534 \text{ ft})}{2}$$

$$p_a = 0.46534 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.00034308 \text{ kip/ft}^2)}{(0.46534 \text{ kip/ft}^2)}$$

$$\text{Ratio} = -0.00073726$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 1.3 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

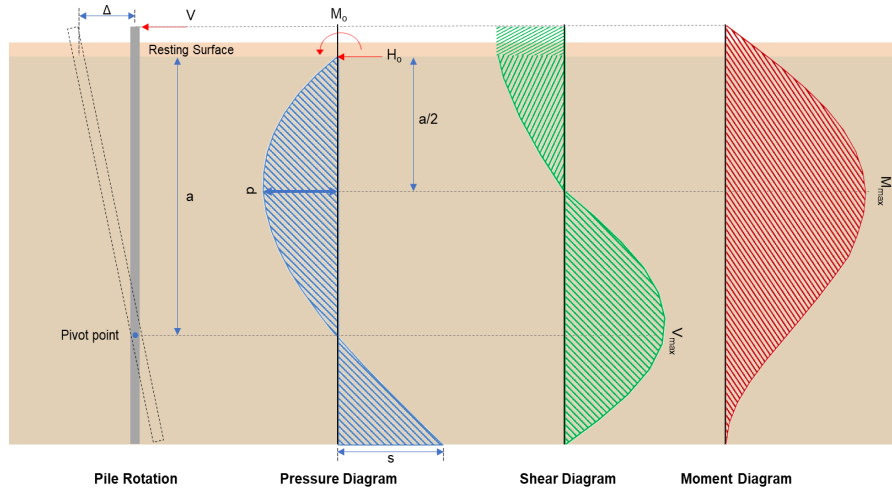
Status: **PASS**
Ratio: **0.000**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(-0.000097596 \text{ kip/ft}^2)}{(1.3 \text{ kip/ft}^2)}$$

$$Ratio = -0.000075074$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.53313 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.992 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 6.624 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.624 \text{ kipft/ft})}{(-0.53313 \text{ kip/ft})}$$

$$E = 12.425 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.624 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.53313 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.624 \text{ kipft/ft})) + (4 \times (-0.53313 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4734 \text{ ft}$$

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

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

$$V_{max} = ((-0.53313 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4734 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4734 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.241 \text{ kip}$$

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

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

$$M_{max} = ((-0.53313 \text{ kip/ft}) \times (96 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(12.425 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 53.512 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.012 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.0015 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.004375 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.004375 \text{ kipft/ft})}{(-0.0015 \text{ kip/ft})}$$

$$E = 2.9167 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.004375 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.0015 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.004375 \text{ kipft/ft})) + (4 \times (-0.0015 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6571 \text{ ft}$$

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

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

$$V_{max} = ((-0.0015 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6571 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6571 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.017537 \text{ kip}$$

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

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

$$M_{max} = ((-0.0015 \text{ kip/ft}) \times (96 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.9167 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050047 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 7238.2 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.285 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (7238.2 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (7238.2 \text{ in}^2)) \right]$$

$$A_{st,required} = -265.35 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-265.35 \text{ in}^2), (0.0018 \times (7238.2 \text{ in}^2))]$$

$$A_{min} = 13.029 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(13.029 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 44$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (44) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 13.499 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.029 \text{ in}^2)}{(13.499 \text{ in}^2)}$$

$$\text{Ratio} = 0.96517$$

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 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (96 \text{ in})]$$

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

Summary:

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

Main reinforcement: **44 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(7238.2 \text{ in}^2) - (13.499 \text{ in}^2)]) + ((60 \text{ ksi}) \times (13.499 \text{ in}^2))]$$

$$\phi P_N = 8929.8 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13,285 \text{ kip})}{(8929.8 \text{ kip})}$$

$$\text{Ratio} = 0.0014877$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

$b_w = 96 \text{ in}$ - Effective width,
22.5.2.2 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (96 \text{ in})$$

$$d = 76.8 \text{ in}$$

22.5.5.1.3 λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(76.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.48002$$

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

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

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

$$V_{c,max} = 5 \times (0.48002) \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,max} = 884.76 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.285 \text{ kip} \rightarrow 13285 \text{ lbf}$.

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

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

$$V_{c,a} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + \frac{(13285 \text{ lbf})}{6 \times (7238.2 \text{ in}^2)} \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,a} = 356.16 \text{ kip}$$

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

22.5.5.1.2 $V_{c,b}$ - Shear strength of concrete (b)

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

$$V_{c,b} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,b} = 1275.5 \text{ kip}$$

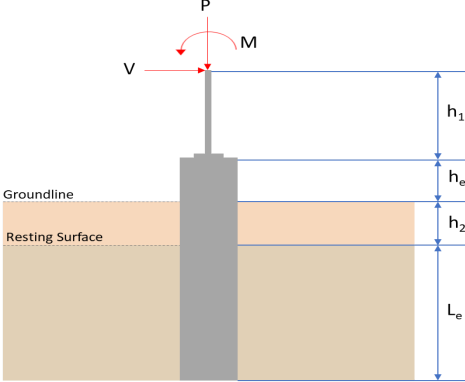
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(884.76 \text{ kip}), (356.16 \text{ kip}), (1275.5 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 356.16 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$ $V_{s,a} = 2949.1 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (76.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 101.79 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(2949.1 \text{ kip}), (101.79 \text{ kip})]$ $V_s = 101.79 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((356.16 \text{ kip}) + (101.79 \text{ kip}))$ $\phi V_n = 297.67 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 17.241 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(17.241 \text{ kip})}{(297.67 \text{ kip})}$ $Ratio = 0.05792$ <p>Considering z-direction:</p> <p>$V_{max} = 0.017537 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.017537 \text{ kip})}{(297.67 \text{ kip})}$ $Ratio = 0.000058914$	<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> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (96 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 86859 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 86858.753 \text{ in}^3$ $\phi M_{n,1} = 1176.212 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (86859 \text{ in}^3)$ $\phi M_{n,2} = 9997.8 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(1176.2 \text{ kipft}), (9997.8 \text{ kipft})]$ $\phi M_n = 1176.2 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 53.512 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(53.512 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.045496$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.050047 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.050047 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.000042549$	<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: round $D = 96 \text{ in}$ - Pile diameter $L = 6.5 \text{ ft}$ - Total pile length $h_1 = 0 \text{ ft}$ - Lateral load height from the top of the pile, $h_2 = 0 \text{ ft}$ - Depth to resisting surface $h_e = 0 \text{ ft}$ - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="517 1079 1094 1169"> <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>Sandy gravel and/or gravel</td> <td>3000.000</td> <td>200.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1263 935 1438"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.266</td> <td>13.285</td> </tr> <tr> <td>V_x (kip)</td> <td>-2.555</td> <td>-4.265</td> </tr> <tr> <td>V_z (kip)</td> <td>0.007</td> <td>0.012</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.021</td> <td>0.035</td> </tr> <tr> <td>M_z (kipft)</td> <td>31.427</td> <td>52.992</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)	1	Sandy gravel and/or gravel	3000.000	200.000	Load Component	ASD	LRFD	P (kip)	8.266	13.285	V_x (kip)	-2.555	-4.265	V_z (kip)	0.007	0.012	M_x (kipft)	0.021	0.035	M_z (kipft)	31.427	52.992	
Layer	Label	Allowable Bearing Pressure (q_a) (psf)	Allowable Lateral Pressure (R) (psf/ft)																									
1	Sandy gravel and/or gravel	3000.000	200.000																									
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V_z (kip)	0.007	0.012																										
M_x (kipft)	0.021	0.035																										
M_z (kipft)	31.427	52.992																										
	<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> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-2.555 \text{ kip})}{(96 \text{ in})}$ $H_o = -0.31938 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$																											

$$M_o = \frac{(31.427 \text{ kipft}) + ((-2.555 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 3.9284 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.141 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.007 \text{ kip})}{(96 \text{ in})}$$

$$H_o = 0.000875 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.021 \text{ kipft}) + ((0.007 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.002625 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.66116 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.141 \text{ ft}), (0.66116 \text{ ft})]$$

$$L_{e,req} = 6.141 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (6.5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.141 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.94477$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(96 \text{ in})}{2}\right)^2$$

$$A = 50.265 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(8.266 \text{ kip})}{(50.265 \text{ ft}^2)}$$

$$q = 0.16445 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.16445 \text{ kip/ft}^2)}{(3000 \text{ psf})}$$

$$\text{Ratio} = 0.054816$$

Status: **PASS**
Ratio: **0.050**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(96 \text{ in})}$$

$$L/D = 0.8125$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.31938 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.9284 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9284 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.9284 \text{ kipft/ft})) + (4 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4744 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (3.9284 \text{ kipft/ft})) + (3 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.9284 \text{ kipft/ft})) + (2 \times (-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.32866 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (3.9284 \text{ kipft/ft})) + ((-0.31938 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 1.2896 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.4744 \text{ ft})}{2}$$

$$p_a = 0.44744 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.32866 \text{ kip/ft}^2)}{(0.44744 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.73453$$

Status: **PASS**
Ratio: **0.730**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 1.3 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(1.2896 \text{ kip/ft}^2)}{(1.3 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.99197$$

Status: **PASS**
Ratio: **0.990**

Considering z-direction:

$H_o = 0.000875 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.002625 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.002625 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.000875 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.002625 \text{ kipft/ft})) + (4 \times (0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6534 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.002625 \text{ kipft/ft})) + (3 \times (0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.002625 \text{ kipft/ft})) + (2 \times (0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.0011003 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.002625 \text{ kipft/ft})) + ((0.000875 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0024399 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (200 \text{ psf/ft}) \times \frac{(4.6534 \text{ ft})}{2}$$

$$p_a = 0.46534 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0011003 \text{ kip/ft}^2)}{(0.46534 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0023646$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (200 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 1.3 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

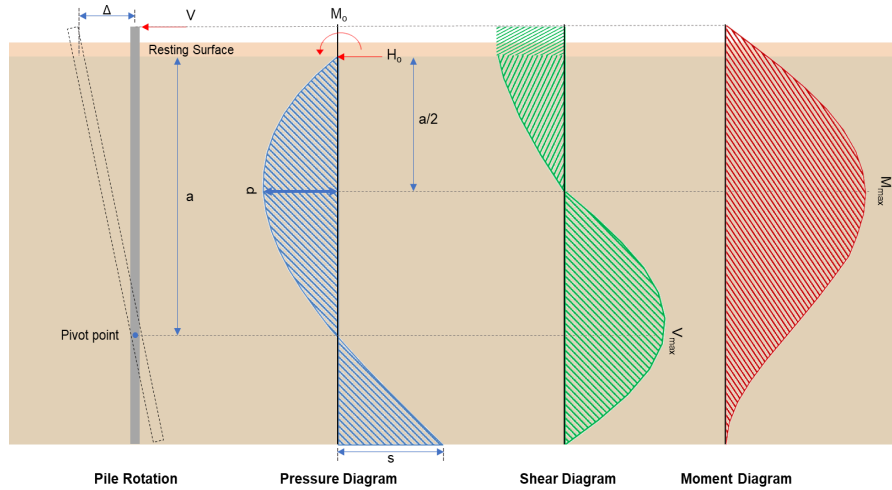
Status: **PASS**
Ratio: **0.000**

$$ratio = \frac{M_o}{p_s}$$

$$Ratio = \frac{(0.0024399 \text{ kip/ft}^2)}{(1.3 \text{ kip/ft}^2)}$$

$$Ratio = 0.0018768$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.265 \text{ kip})}{(96 \text{ in})}$$

$$H_o = -0.53313 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(52.992 \text{ kipft}) + ((-4.265 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 6.624 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.624 \text{ kipft/ft})}{(-0.53313 \text{ kip/ft})}$$

$$E = 12.425 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.624 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.53313 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.624 \text{ kipft/ft})) + (4 \times (-0.53313 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4734 \text{ ft}$$

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

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

$$V_{max} = ((-0.53313 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4734 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4734 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 17.241 \text{ kip}$$

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

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

$$M_{max} = ((-0.53313 \text{ kip/ft}) \times (96 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(12.425 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (12.425 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.4734 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 53.512 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.012 \text{ kip})}{(96 \text{ in})}$$

$$H_o = 0.0015 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{(96 \text{ in})}$$

$$M_o = 0.004375 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.004375 \text{ kipft/ft})}{(0.0015 \text{ kip/ft})}$$

$$E = 2.9167 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.004375 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.0015 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.004375 \text{ kipft/ft})) + (4 \times (0.0015 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.6571 \text{ ft}$$

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

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

$$V_{max} = ((0.0015 \text{ kip/ft}) \times (96 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6571 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6571 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.017537 \text{ kip}$$

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

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

$$M_{max} = ((0.0015 \text{ kip/ft}) \times (96 \text{ in}) \times (6.5 \text{ ft})) \times \left[\left(\frac{(2.9167 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left(\frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9167 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left(\frac{(4.6571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.050047 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

- $f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 7238.2 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(13.285 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (2.5 \text{ ksi}) \times (7238.2 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (7238.2 \text{ in}^2)) \right]$$

$$A_{st,required} = -265.35 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-265.35 \text{ in}^2), (0.0018 \times (7238.2 \text{ in}^2))]$$

$$A_{min} = 13.029 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(13.029 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 44$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (44) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 13.499 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.029 \text{ in}^2)}{(13.499 \text{ in}^2)}$$

$$\text{Ratio} = 0.96517$$

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 \varnothing : Use #3(0.375 in)

25.7.2.1

s_{ties} - Maximum center-to-center spacing of ties,

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (96 \text{ in})]$$

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

Summary:

Main reinforcement: **44 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times [(0.85 \times (2.5 \text{ ksi}) \times [(7238.2 \text{ in}^2) - (13.499 \text{ in}^2)]) + ((60 \text{ ksi}) \times (13.499 \text{ in}^2))]$$

$$\phi P_N = 8929.8 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.285 \text{ kip})}{(8929.8 \text{ kip})}$$

$$\text{Ratio} = 0.0014877$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

22.5.2.2

$b_w = 96 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (96 \text{ in})$$

$$d = 76.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(76.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.48002$$

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

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.48002) \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,max} = 884.76 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 13.285 \text{ kip} \rightarrow 13285 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + \frac{(13285 \text{ lbf})}{6 \times (7238.2 \text{ in}^2)} \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,a} = 356.16 \text{ kip}$$

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

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.48002) \times \sqrt{(2500 \text{ psi})} + (0.05 \times (2500 \text{ psi})) \right] \times (96 \text{ in}) \times (76.8 \text{ in})$$

$$V_{c,b} = 1275.5 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(884.76 \text{ kip}), (356.16 \text{ kip}), (1275.5 \text{ kip})]$$

<p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p>	<p style="text-align: center;">$V_c = 356.16 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (96 \text{ in}) \times (76.8 \text{ in})$ $V_{s,a} = 2949.1 \text{ kip}$ <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>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (76.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 101.79 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(2949.1 \text{ kip}), (101.79 \text{ kip})]$ $V_s = 101.79 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((356.16 \text{ kip}) + (101.79 \text{ kip}))$ $\phi V_n = 297.67 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 17.241 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(17.241 \text{ kip})}{(297.67 \text{ kip})}$ $Ratio = 0.05792$ <p>Considering z-direction:</p> <p>$V_{max} = 0.017537 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.017537 \text{ kip})}{(297.67 \text{ kip})}$ $Ratio = 0.000058914$	<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> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (96 \text{ in})^3}{32}$	

<p>14.5.2.1b</p>	<p style="text-align: center;">$S_m = 86859 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 86858.753 \text{ in}^3$ $\phi M_{n,1} = 1176.212 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (86859 \text{ in}^3)$ $\phi M_{n,2} = 9997.8 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(1176.2 \text{ kipft}), (9997.8 \text{ kipft})]$ $\phi M_n = 1176.2 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 53.512 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(53.512 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.045496$	<p>Status: PASS Ratio: 0.050</p>
	<p>Considering z-direction: $M_{max} = 0.050047 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.050047 \text{ kipft})}{(1176.2 \text{ kipft})}$ $\text{Ratio} = 0.000042549$	<p>Status: PASS Ratio: 0.000</p>