

# Your Project Calculations



Project Name: ACFG2

S3D Model Link:

[https://platform.skyciv.com/structural?preload\\_name=ACFG2&preload\\_path=Shared%20Enterprise%20Folder/MT\\_Solar\\_Projects/6\\_2023](https://platform.skyciv.com/structural?preload_name=ACFG2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023)

Public Model Link:

[https://platform.skyciv.com/structural-viewer?project\\_id=iYaa76Pb5qDhtBThxfPXfRh6Q4H2F5vJQX3YqAIBZYJHOroco1GlvUEysssBmjy](https://platform.skyciv.com/structural-viewer?project_id=iYaa76Pb5qDhtBThxfPXfRh6Q4H2F5vJQX3YqAIBZYJHOroco1GlvUEysssBmjy)

## Array Specification

<b>Product:</b>	Beam
<b>Unique ID:</b>	5P-19.75-8TOP-XD-45-L-4Hx15W-B38E
<b>Duty Classification:</b>	XD
<b>Module Width:</b>	41.00 in
<b>Module Length:</b>	74.00in
<b>Number of Rows:</b>	4
<b>Number of Columns:</b>	15
<b>Total Number of Modules:</b>	60
<b>Desired Tilt Angle:</b>	20
<b>Front Edge Clearance:</b>	15
<b>Total Array Height at Tilt:</b>	19.70 ft
<b>Total Frame Length:</b>	94.00 ft
<b>Frame Weight:</b>	6099 lbs
<b>Array Dimensions N/S:</b>	13.83 ft
<b>Array Dimensions E/W:</b>	93.75 ft
<b>Rail Length:</b>	166.00 in
<b>Rail Spacing:</b>	3.08 ft
<b>Rail Check:</b>	Not Checked

## Support Specifications

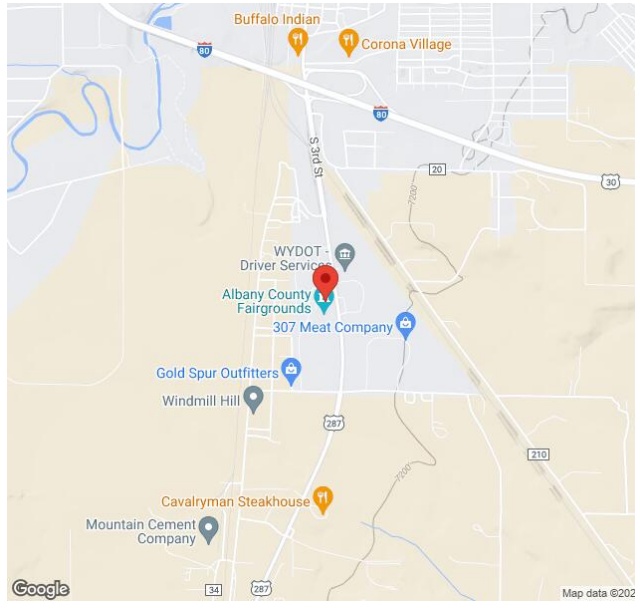
<b>Pole Size:</b>	8in Pipe Sch 40
<b>Pole Length above Grade:</b>	17.37 ft
<b>Number of Poles:</b>	5
<b>Pole Spacing:</b>	19.75 ft

## Foundation Specifications

<b>Foundation Type:</b>	Square
<b>Foundation Dimensions:</b>	48 x 48 in
<b>Foundation Depth (below grade):</b>	Pile 1: 6.50 ft Pile 2: 6.50 ft Pile 3: 6.75 ft Pile 4: 6.50 ft Pile 5: 6.50 ft
<b>Foundation Volume:</b>	19.407 y <sup>3</sup>
<b>Foundation Result:</b>	PASSED
<b>Mount Twist:</b>	0.187662 kip

## Site Info

<b>Risk Category:</b>	I
<b>Exposure:</b>	C
<b>Soil Classification:</b>	sand
<b>Site Location:</b>	3510 S 3rd St, Laramie, WY 82070, USA
<b>Wind Speed:</b>	110 mph
<b>Snow Load:</b>	30 psf
<b>Design Uplift Pressure:</b>	Multiple pressures
<b>Design Downforce Pressure:</b>	Multiple pressures
<b>Design Snow Pressure:</b>	0.016495 ksf



### Design Disclaimer

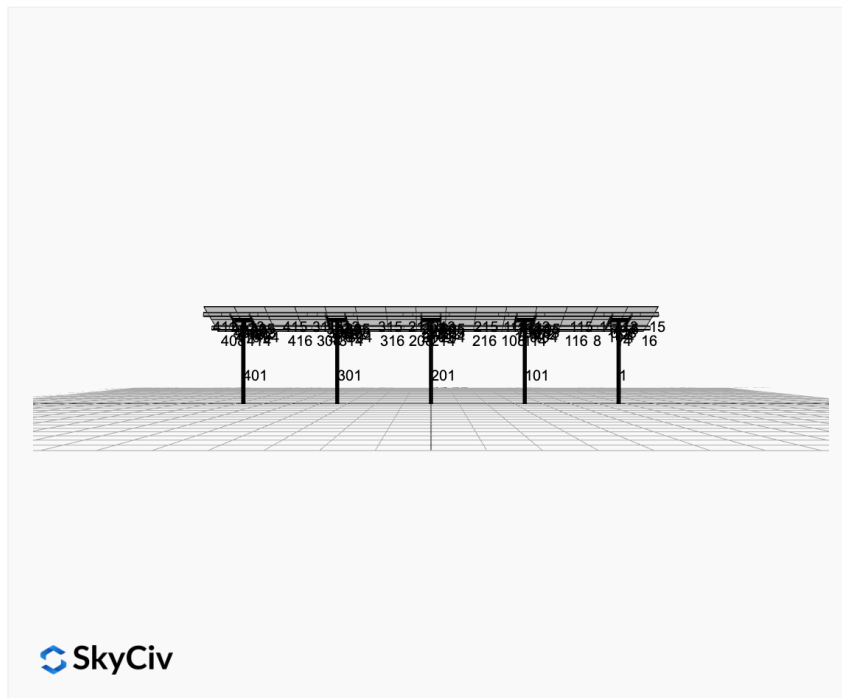
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

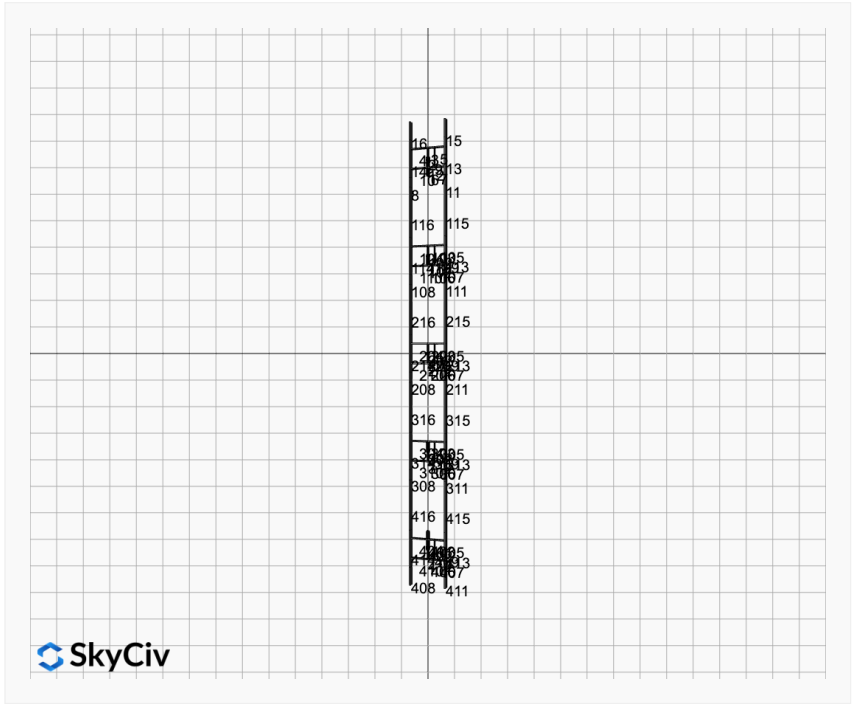
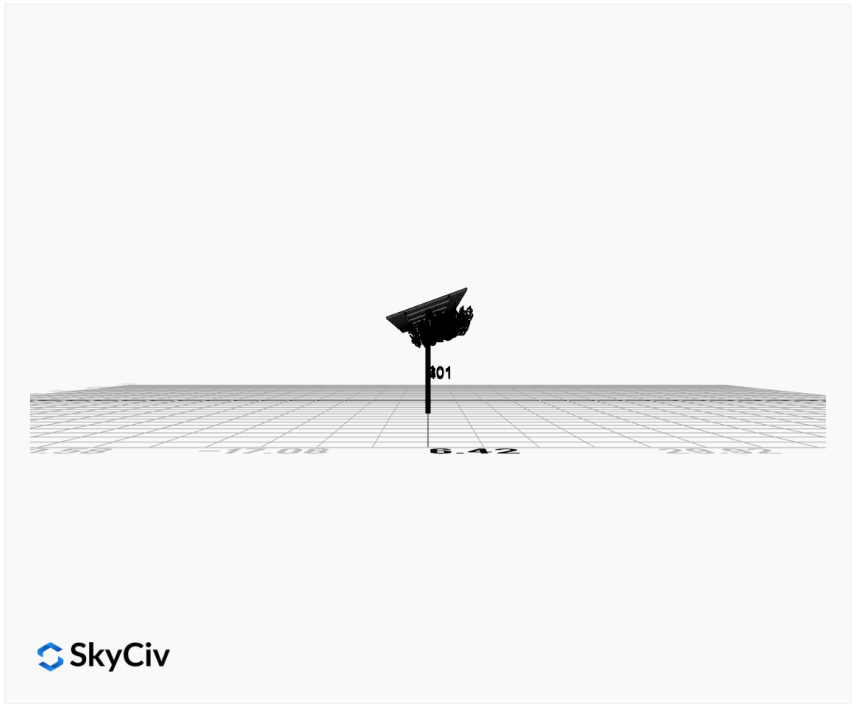
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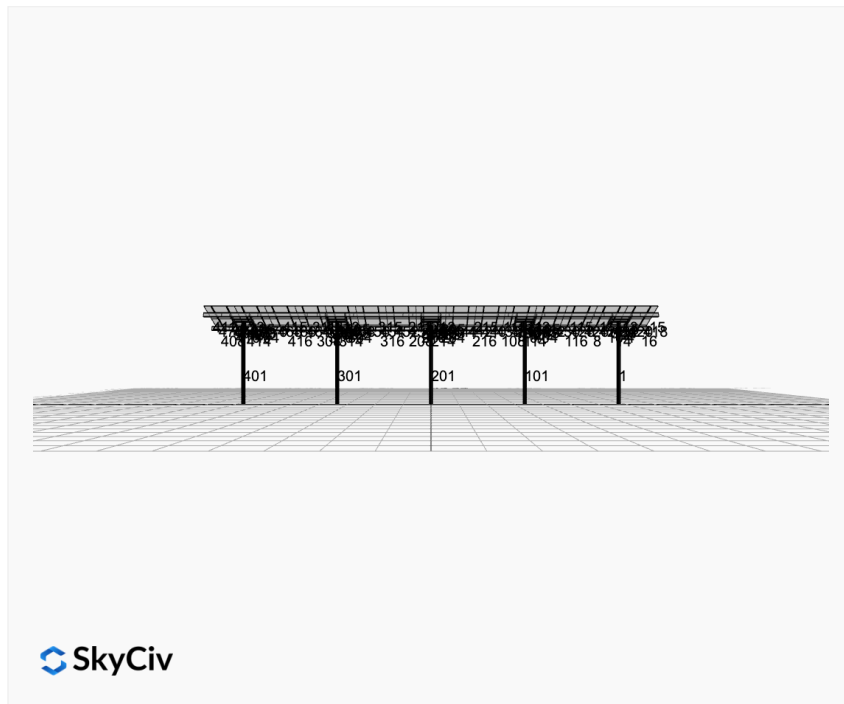
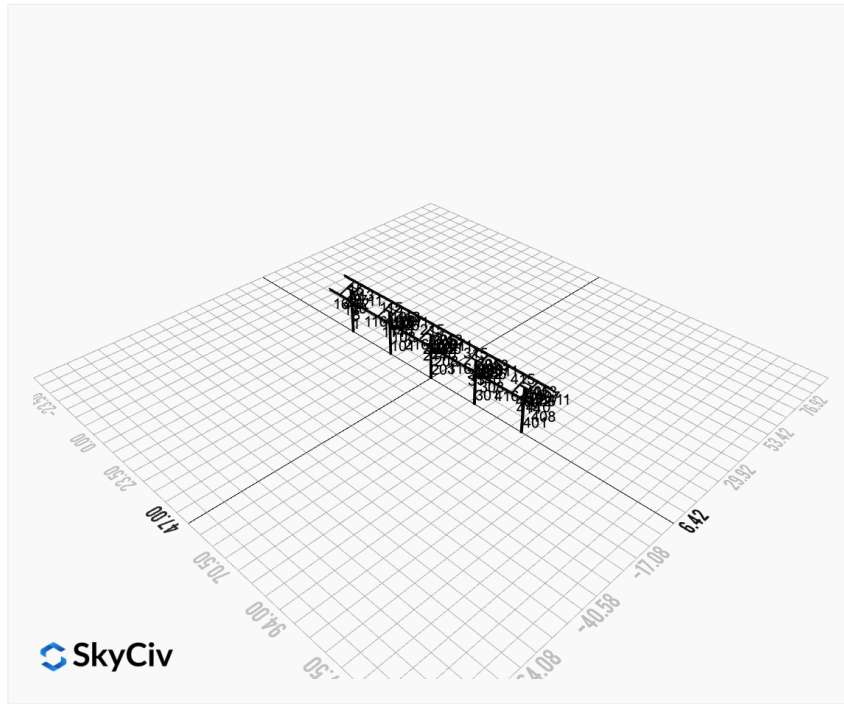
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### Design Notes:

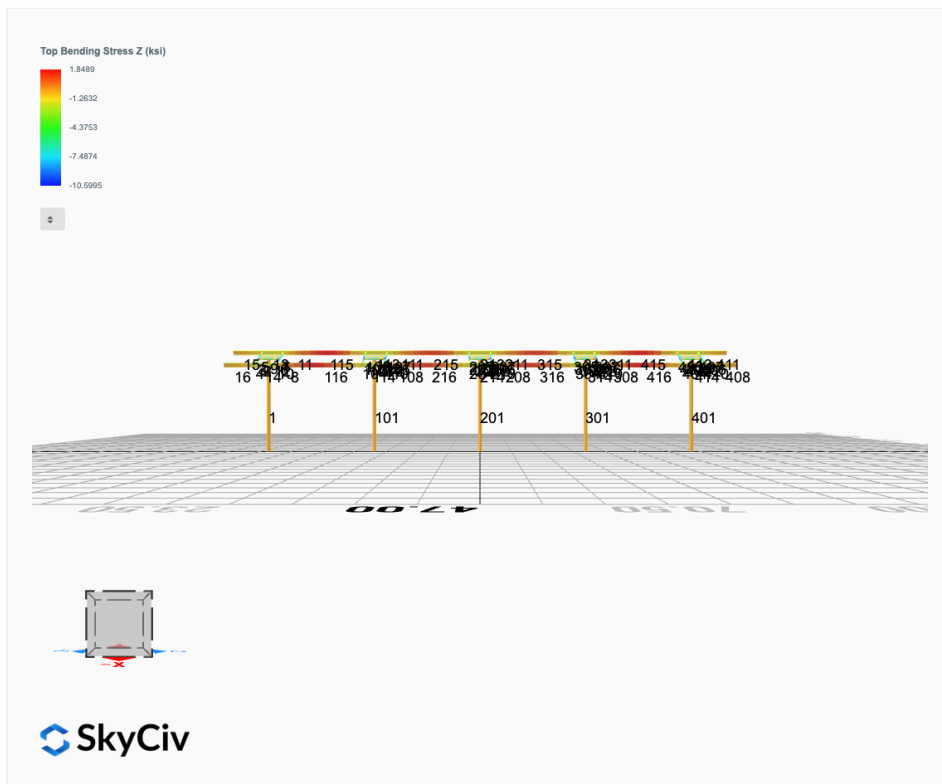
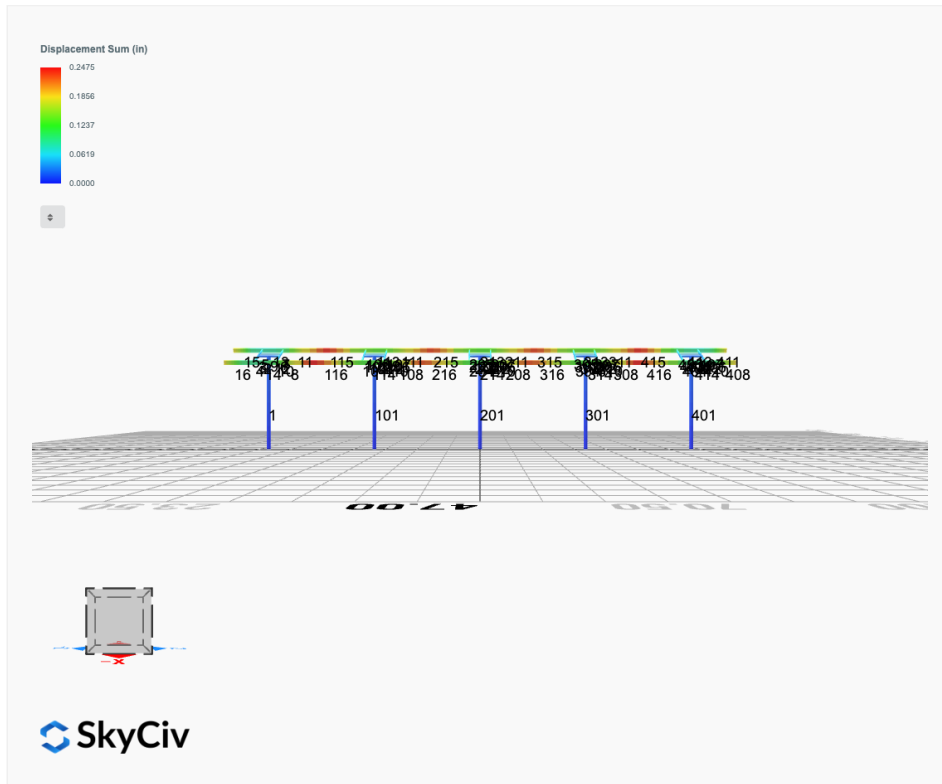
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only

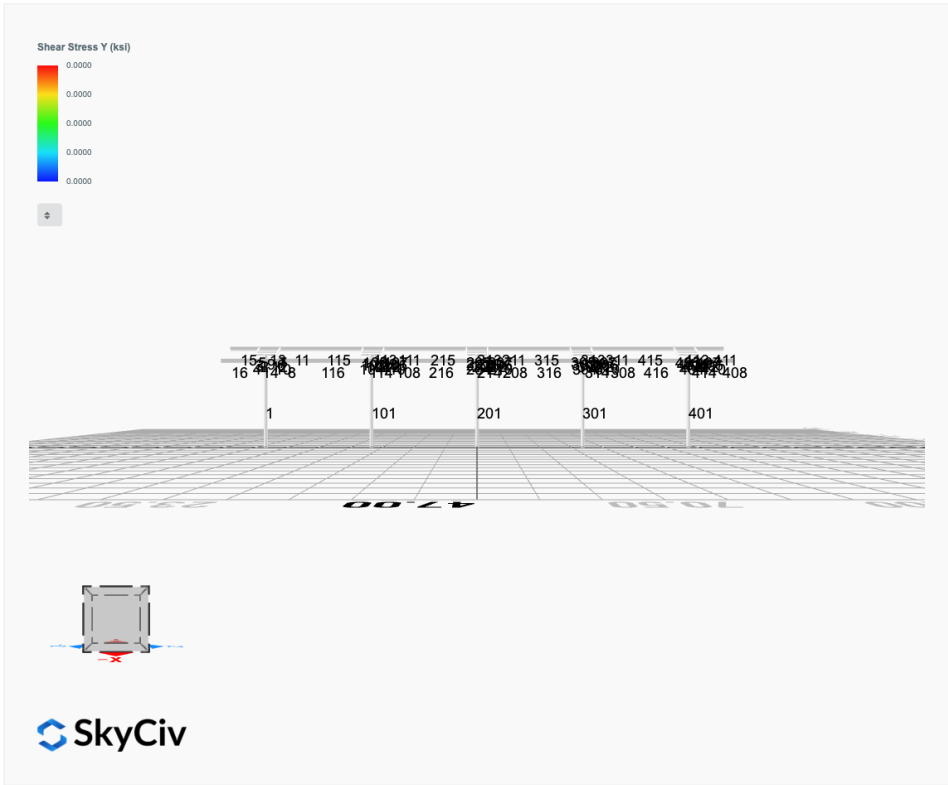
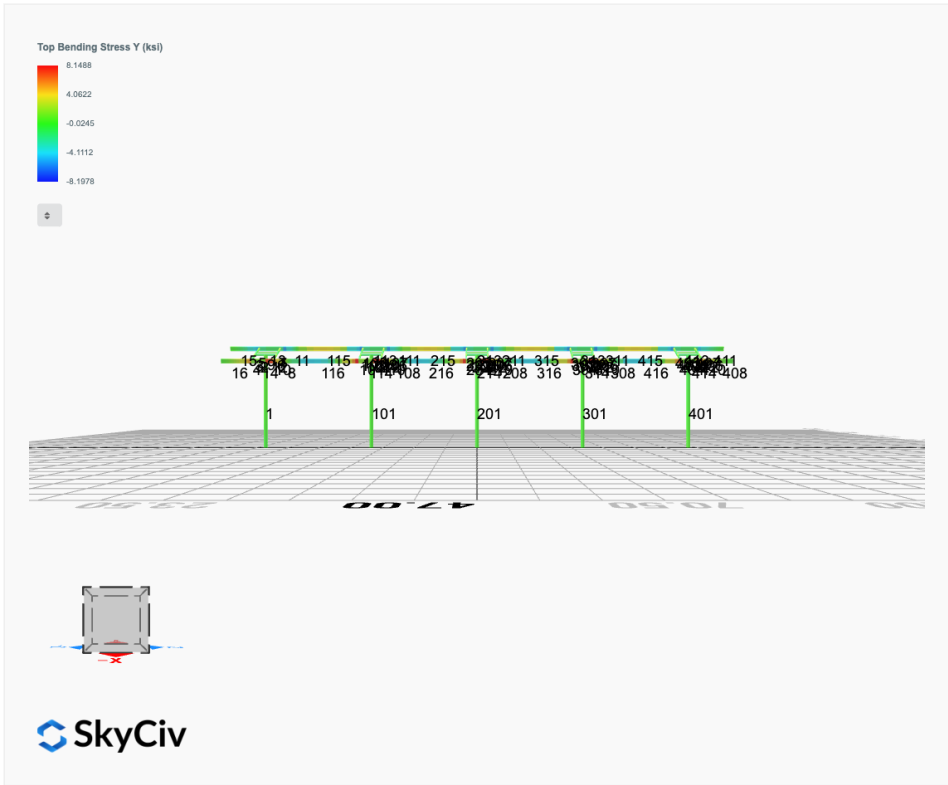






## FEM Results (Envelope Worst Case for each member)

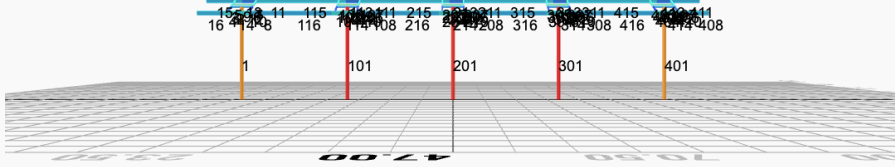




Axial Stress (ksi)



σ



SkyCiv

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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0042	2.2941	0.0146	0.0827	-0.0023	-0.0346
ULS: 2. D + L	0.0042	2.2941	0.0146	0.0827	-0.0023	-0.0346
ULS: 3. D + (S or Lr or R)	0.0143	5.9788	0.0503	0.2848	-0.0085	-0.1876
ULS: 3. D + (S or Lr or R)	0.0042	2.2941	0.0146	0.0827	-0.0023	-0.0346
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0117	5.0576	0.0414	0.2343	-0.0069	-0.1493
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	2.2941	0.0146	0.0827	-0.0023	-0.0346
ULS: 5b. D + 0.7E	0.0042	2.2941	0.0146	0.0827	-0.0023	-0.0346
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0117	5.0576	0.0414	0.2343	-0.0069	-0.1493
ULS: 8. 0.6D + 0.7E	0.0025	1.3764	0.0088	0.0496	-0.0014	-0.0208
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.2477	5.6382	0.0666	0.3724	-0.1078	23.4229
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.2477	5.6382	0.0666	0.3724	-0.1078	23.4229
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0588	-0.5396	-0.0253	-0.1382	0.0768	-17.0750
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9401	-0.1777	-0.0303	-0.1661	0.0927	-23.8890
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9272	7.5657	0.0804	0.4516	-0.0861	17.4439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9272	7.5657	0.0804	0.4516	-0.0861	17.4439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8027	2.9323	0.0115	0.0686	0.0524	-12.9296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7137	3.2038	0.0077	0.0477	0.0643	-18.0401
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9347	4.8021	0.0536	0.3000	-0.0814	17.5585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9347	4.8021	0.0536	0.3000	-0.0814	17.5585
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7952	0.1688	-0.0153	-0.0830	0.0570	-12.8149
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7062	0.4402	-0.0191	-0.1039	0.0689	-17.9254
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.2494	4.7205	0.0608	0.3393	-0.1069	23.4368
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.2494	4.7205	0.0608	0.3393	-0.1069	23.4368
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0572	-1.4572	-0.0311	-0.1713	0.0777	-17.0611
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9385	-1.0953	-0.0362	-0.1992	0.0936	-23.8752

### 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	11.4367
Shear X	-2.0865
Shear Z	0.1245
Moment X	0.6987
Moment Y (Twist)	0.1877
Moment Z	41.4468

### 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	7.5657
Shear X	-1.2494
Shear Z	0.0804
Moment X	0.4516
Moment Y (Twist)	0.1078
Moment Z	23.8890

## Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0037	2.5380	-0.0006	-0.0036	0.0004	0.0855
ULS: 2. D + L	-0.0037	2.5380	-0.0006	-0.0036	0.0004	0.0855
ULS: 3. D + (S or Lr or R)	-0.0125	6.8120	-0.0021	-0.0123	0.0012	0.2291
ULS: 3. D + (S or Lr or R)	-0.0037	2.5380	-0.0006	-0.0036	0.0004	0.0855
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0103	5.7435	-0.0017	-0.0101	0.0010	0.1932
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	2.5380	-0.0006	-0.0036	0.0004	0.0855
ULS: 5b. D + 0.7E	-0.0037	2.5380	-0.0006	-0.0036	0.0004	0.0855

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0103	5.7435	-0.0017	-0.0101	0.0010	0.1932
ULS: 8. 0.6D + 0.7E	-0.0022	1.5228	-0.0004	-0.0021	0.0002	0.0513
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3754	6.3828	0.0106	0.0576	-0.0402	25.7171
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3754	6.3828	0.0106	0.0576	-0.0402	25.7171
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1662	-0.7289	-0.0071	-0.0387	0.0256	-18.5516
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9978	-0.2897	-0.0133	-0.0727	0.0449	-25.6033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0391	8.6271	0.0066	0.0358	-0.0295	19.4169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0391	8.6271	0.0066	0.0358	-0.0295	19.4169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8671	3.2934	-0.0066	-0.0365	0.0199	-13.7846
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7408	3.6227	-0.0113	-0.0620	0.0343	-19.0733
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0325	5.4216	0.0078	0.0423	-0.0301	19.3092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0325	5.4216	0.0078	0.0423	-0.0301	19.3092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8737	0.0878	-0.0055	-0.0299	0.0193	-13.8923
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7474	0.4172	-0.0101	-0.0554	0.0337	-19.1811
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3740	5.3676	0.0108	0.0591	-0.0404	25.6829
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3740	5.3676	0.0108	0.0591	-0.0404	25.6829
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1677	-1.7440	-0.0069	-0.0373	0.0255	-18.5858
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9993	-1.3049	-0.0130	-0.0713	0.0447	-25.6374

### 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	13.0858
Shear X	-2.2914
Shear Z	-0.0243
Moment X	-0.1330
Moment Y (Twist)	0.0802
Moment Z	44.5754

### 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	8.6271
Shear X	-1.3754
Shear Z	-0.0133
Moment X	-0.0727
Moment Y (Twist)	0.0449
Moment Z	25.7171

### 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	-0.0010	2.5229	-0.0000	0.0000	0.0000	0.0575
ULS: 2. D + L	-0.0010	2.5229	-0.0000	0.0000	0.0000	0.0575
ULS: 3. D + (S or Lr or R)	-0.0035	6.7603	0.0000	-0.0000	0.0000	0.1332
ULS: 3. D + (S or Lr or R)	-0.0010	2.5229	-0.0000	0.0000	0.0000	0.0575
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0029	5.7010	0.0000	0.0000	0.0000	0.1143
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0010	2.5229	-0.0000	0.0000	0.0000	0.0575
ULS: 5b. D + 0.7E	-0.0010	2.5229	-0.0000	0.0000	0.0000	0.0575
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0029	5.7010	0.0000	0.0000	0.0000	0.1143
ULS: 8. 0.6D + 0.7E	-0.0006	1.5138	-0.0000	0.0000	0.0000	0.0345
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3866	6.3687	-0.0000	0.0000	0.0000	26.0683
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3866	6.3687	-0.0000	0.0000	0.0000	26.0683
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1779	-0.7385	-0.0000	0.0000	0.0000	-18.8010
ULS: 5a. D + 0.6W_Wind uplift Case B only	1.0150	-0.3158	-0.0000	0.0000	0.0000	-26.0663
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0421	8.5853	0.0000	0.0000	0.0000	19.6224
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0421	8.5853	0.0000	0.0000	0.0000	19.6224
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8813	3.2549	0.0000	0.0000	0.0000	-14.0296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7592	3.5719	0.0000	0.0000	0.0000	-19.4786

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	-1.0402	5.4073	-0.0000	0.0000	0.0000	19.5656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0402	5.4073	-0.0000	0.0000	0.0000	19.5656
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8831	0.0768	-0.0000	0.0000	0.0000	-14.0864
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7610	0.3939	-0.0000	0.0000	0.0000	-19.5354
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3862	5.3596	-0.0000	0.0000	0.0000	26.0453
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3862	5.3596	-0.0000	0.0000	0.0000	26.0453
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1783	-1.7477	-0.0000	0.0000	0.0000	-18.8240
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	1.0154	-1.3250	-0.0000	0.0000	0.0000	-26.0893

### 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	13.0136
Shear X	-2.3101
Shear Z	0.0000
Moment X	0.0002
Moment Y (Twist)	0.0001
Moment Z	45.2300

### 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	8.5853
Shear X	-1.3866
Shear Z	-0.0000
Moment X	0.0000
Moment Y (Twist)	0.0000
Moment Z	26.0893

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

#### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	-0.0037	2.5380	0.0006	0.0036	-0.0003	0.0855
ULS: 2. D + L	-0.0037	2.5380	0.0006	0.0036	-0.0003	0.0855
ULS: 3. D + (S or Lr or R)	-0.0125	6.8120	0.0021	0.0123	-0.0011	0.2291
ULS: 3. D + (S or Lr or R)	-0.0037	2.5380	0.0006	0.0036	-0.0003	0.0855
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0103	5.7435	0.0017	0.0101	-0.0009	0.1932
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	-0.0037	2.5380	0.0006	0.0036	-0.0003	0.0855
ULS: 5b. D + 0.7E	-0.0037	2.5380	0.0006	0.0036	-0.0003	0.0855
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	-0.0103	5.7435	0.0017	0.0101	-0.0009	0.1932
ULS: 8. 0.6D + 0.7E	-0.0022	1.5228	0.0004	0.0022	-0.0002	0.0513
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.3754	6.3828	-0.0106	-0.0576	0.0402	25.7171
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.3754	6.3828	-0.0106	-0.0576	0.0402	25.7171
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.1662	-0.7289	0.0071	0.0387	-0.0256	-18.5516
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9978	-0.2897	0.0133	0.0727	-0.0449	-25.6033
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0391	8.6271	-0.0066	-0.0358	0.0295	19.4169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0391	8.6271	-0.0066	-0.0358	0.0295	19.4169
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8671	3.2934	0.0066	0.0365	-0.0199	-13.7846
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7408	3.6227	0.0113	0.0620	-0.0343	-19.0733
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-1.0325	5.4216	-0.0078	-0.0423	0.0301	19.3092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-1.0325	5.4216	-0.0078	-0.0423	0.0301	19.3092
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8737	0.0878	0.0055	0.0299	-0.0193	-13.8923
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7474	0.4172	0.0101	0.0554	-0.0337	-19.1811
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.3740	5.3676	-0.0108	-0.0591	0.0404	25.6829
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.3740	5.3676	-0.0108	-0.0591	0.0404	25.6829
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.1677	-1.7440	0.0069	0.0373	-0.0255	-18.5858
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9993	-1.3049	0.0130	0.0713	-0.0447	-25.6374

### 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	13.0858
Shear X	-2.2914
Shear Z	0.0243
Moment X	0.1333
Moment Y (Twist)	0.0801
Moment Z	44.5756

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	8.6271
Shear X	-1.3754
Shear Z	0.0133
Moment X	0.0727
Moment Y (Twist)	0.0449
Moment Z	25.7171

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

### ASD Load Combination Results

Name	Fx	Fy	Fz	Mx	My	Mz
ULS: 1. D	0.0042	2.2941	-0.0146	-0.0827	0.0023	-0.0346
ULS: 2. D + L	0.0042	2.2941	-0.0146	-0.0827	0.0023	-0.0346
ULS: 3. D + (S or Lr or R)	0.0143	5.9788	-0.0503	-0.2848	0.0085	-0.1875
ULS: 3. D + (S or Lr or R)	0.0042	2.2941	-0.0146	-0.0827	0.0023	-0.0346
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0117	5.0576	-0.0414	-0.2343	0.0069	-0.1493
ULS: 4. D + 0.75L + 0.75(S or Lr or R)	0.0042	2.2941	-0.0146	-0.0827	0.0023	-0.0346
ULS: 5b. D + 0.7E	0.0042	2.2941	-0.0146	-0.0827	0.0023	-0.0346
ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S	0.0117	5.0576	-0.0414	-0.2343	0.0069	-0.1493
ULS: 8. 0.6D + 0.7E	0.0025	1.3764	-0.0088	-0.0496	0.0014	-0.0208
ULS: 5a. D + 0.6W_Wind downforce Case A only	-1.2477	5.6382	-0.0666	-0.3724	0.1078	23.4229
ULS: 5a. D + 0.6W_Wind downforce Case B only	-1.2477	5.6382	-0.0666	-0.3724	0.1078	23.4229
ULS: 5a. D + 0.6W_Wind uplift Case A only	1.0588	-0.5396	0.0253	0.1382	-0.0768	-17.0750
ULS: 5a. D + 0.6W_Wind uplift Case B only	0.9401	-0.1777	0.0303	0.1661	-0.0927	-23.8890
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9272	7.5657	-0.0804	-0.4516	0.0861	17.4439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9272	7.5657	-0.0804	-0.4516	0.0861	17.4439
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.8027	2.9323	-0.0115	-0.0686	-0.0524	-12.9296
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7137	3.2038	-0.0077	-0.0477	-0.0643	-18.0401
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only	-0.9347	4.8021	-0.0536	-0.3000	0.0814	17.5586
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only	-0.9347	4.8021	-0.0536	-0.3000	0.0814	17.5586
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only	0.7952	0.1688	0.0153	0.0830	-0.0570	-12.8149
ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only	0.7062	0.4402	0.0191	0.1039	-0.0689	-17.9254
ULS: 7. 0.6D + 0.6W_Wind downforce Case A only	-1.2494	4.7205	-0.0608	-0.3393	0.1069	23.4368
ULS: 7. 0.6D + 0.6W_Wind downforce Case B only	-1.2494	4.7205	-0.0608	-0.3393	0.1069	23.4368
ULS: 7. 0.6D + 0.6W_Wind uplift Case A only	1.0572	-1.4572	0.0311	0.1713	-0.0777	-17.0611
ULS: 7. 0.6D + 0.6W_Wind uplift Case B only	0.9385	-1.0953	0.0362	0.1992	-0.0936	-23.8752

### 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	11.4367
Shear X	-2.0865
Shear Z	-0.1245
Moment X	-0.6987
Moment Y (Twist)	0.1877
Moment Z	41.4475

### 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	7.5657
Shear X	-1.2494
Shear Z	-0.0804
Moment X	-0.4516
Moment Y (Twist)	0.1078
Moment Z	23.8890

## Project Details

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



## Design Input Information

Design Factors			
$\Phi_t$	$\Phi_c$	$\Phi_b$	$\Phi_v$
0.9	0.9	0.9	0.9

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

### Section Dimensions



ID	Name	d (in)	$t_w$ (in)				
3	2in Pipe Sch 120	2.38	0.25				
6	4in Pipe Sch 120	4.50	0.44				
9	8in Pipe Sch 40	8.63	0.32				



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



ID	Name	d (in)	$t_w$ (in)	$b_t$ (in)	$b_b$ (in)	$t_t$ (in)	$t_b$ (in)	r (in)
20	W10x12	9.87	0.19	3.96	3.96	0.21	0.21	0.30

### Section Properties

ID	Name	A (in <sup>2</sup> )	J (in <sup>4</sup> )	$I_{yp}$ (in <sup>4</sup> )	$I_{zp}$ (in <sup>4</sup> )	$I_w$ (in <sup>6</sup> )	$S_{yp}$ (in <sup>3</sup> )	$S_{zp}$ (in <sup>3</sup> )
3	2in Pipe Sch 120	1.67	1.91	0.96	0.96	0.00	1.13	1.13
6	4in Pipe Sch 120	5.58	23.29	11.64	11.64	0.00	7.24	7.24
9	8in Pipe Sch 40	8.40	144.98	72.49	72.49	0.00	22.21	22.21







412	6	4.20	4.20	2.00	-	000	000	1
413	20	4.88	4.00	7.50	1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.11,1.06,1.11,1.10,1.10,1.09,1.11,1.11,1.11,1.13,1.13,1.11,1.11,1.06,1.11,1.10,1.10,1.09,1.11	300	200	1
414	20	4.88	4.00	7.50	1.11,1.12,1.11,1.12,1.12,1.11,1.11,1.11,1.11,1.18,1.11,1.11,1.11,1.65,1.11,1.11,1.12,1.13,1.11,1.11,1.11,1.18,1.11,1.11,1.11,2.63	300	200	1
415	20	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.13,1.13,1.16,1.15,1.13,1.13,1.14,1.14,1.12,1.12,1.11,1.11,1.11,3.1.13,1.16,1.15,1.13,1.13,1.14,1.14	300	200	1
416	20	6.63	6.63	10.20	1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.14,1.11,1.11,1.12,1.18,1.12,1.12,1.12,1.12,1.12,2.1.12,1.12,1.14,1.11,1.11,1.12,1.25	300	200	1

## Member Design Capacity

Member ID	$\Phi_t P_n$ (kip)	$\Phi_c P_n$ (kip)	$\Phi_b M_{zn}$ (k-ft)	$\Phi_b M_{yn}$ (k-ft)	$\Phi_v V_{yn}$ (kip)	$\Phi_v V_{zn}$ (kip)
1	377.97	85.51	83.29	83.29	113.39	113.39
2	251.01	229.64	27.16	27.16	75.30	75.30
3	151.65	150.70	20.17	14.14	54.12	28.95
4	151.65	145.15	20.17	14.14	54.12	28.95
5	151.65	149.10	20.17	14.14	54.12	28.95
6	151.65	150.70	20.17	14.14	54.12	28.95
7	151.65	149.10	20.17	14.14	54.12	28.95
8	159.30	142.47	46.90	6.46	56.26	44.91
9	75.10	66.32	4.25	4.25	22.53	22.53
10	151.65	145.15	20.17	14.14	54.12	28.95
11	159.30	142.47	46.90	6.46	56.26	44.91
12	251.01	248.88	27.16	27.16	75.30	75.30
13	159.30	116.35	32.39	6.46	56.26	44.91
14	159.30	116.35	33.92	6.46	56.26	44.91
15	159.30	55.15	46.90	6.46	56.26	44.91
16	159.30	55.15	46.90	6.46	56.26	44.91
101	377.97	85.51	83.29	83.29	113.39	113.39
102	251.01	229.64	27.16	27.16	75.30	75.30
103	151.65	150.70	20.17	14.14	54.12	28.95
104	151.65	145.15	20.17	14.14	54.12	28.95
105	151.65	149.10	20.17	14.14	54.12	28.95
106	151.65	150.70	20.17	14.14	54.12	28.95
107	151.65	149.10	20.17	14.14	54.12	28.95
108	159.30	142.47	46.90	6.46	56.26	44.91
109	75.10	66.32	4.25	4.25	22.53	22.53
110	151.65	145.15	20.17	14.14	54.12	28.95
111	159.30	142.47	46.90	6.46	56.26	44.91
112	251.01	248.88	27.16	27.16	75.30	75.30
113	159.30	116.35	31.48	6.46	56.26	44.91
114	159.30	116.35	31.78	6.46	56.26	44.91
115	159.30	75.13	20.99	6.46	56.26	44.91
116	159.30	75.13	21.38	6.46	56.26	44.91
201	377.97	85.51	83.29	83.29	113.39	113.39
202	251.01	248.88	27.16	27.16	75.30	75.30
203	151.65	150.70	20.17	14.14	54.12	28.95
204	151.65	145.15	20.17	14.14	54.12	28.95
205	151.65	149.10	20.17	14.14	54.12	28.95
206	151.65	150.70	20.17	14.14	54.12	28.95
207	151.65	149.10	20.17	14.14	54.12	28.95

208	159.30	142.47	46.90	6.46	56.26	44.91
209	75.10	66.32	4.25	4.25	22.53	22.53
210	151.65	145.15	20.17	14.14	54.12	28.95
211	159.30	142.47	46.90	6.46	56.26	44.91
212	251.01	248.88	27.16	27.16	75.30	75.30
213	159.30	116.35	31.48	6.46	56.26	44.91
214	159.30	116.35	31.78	6.46	56.26	44.91
215	159.30	75.13	21.57	6.46	56.26	44.91
216	159.30	75.13	22.15	6.46	56.26	44.91
301	377.97	85.51	83.29	83.29	113.39	113.39
302	251.01	248.88	27.16	27.16	75.30	75.30
303	151.65	150.70	20.17	14.14	54.12	28.95
304	151.65	145.15	20.17	14.14	54.12	28.95
305	151.65	149.10	20.17	14.14	54.12	28.95
306	151.65	150.70	20.17	14.14	54.12	28.95
307	151.65	149.10	20.17	14.14	54.12	28.95
308	159.30	142.47	46.90	6.46	56.26	44.91
309	75.10	66.32	4.25	4.25	22.53	22.53
310	151.65	145.15	20.17	14.14	54.12	28.95
311	159.30	142.47	46.90	6.46	56.26	44.91
312	251.01	229.64	27.16	27.16	75.30	75.30
313	159.30	116.35	31.48	6.46	56.26	44.91
314	159.30	116.35	31.78	6.46	56.26	44.91
315	159.30	75.13	21.38	6.46	56.26	44.91
316	159.30	75.13	21.96	6.46	56.26	44.91
401	377.97	85.51	83.29	83.29	113.39	113.39
402	251.01	248.88	27.16	27.16	75.30	75.30
403	151.65	150.70	20.17	14.14	54.12	28.95
404	151.65	145.15	20.17	14.14	54.12	28.95
405	151.65	149.10	20.17	14.14	54.12	28.95
406	151.65	150.70	20.17	14.14	54.12	28.95
407	151.65	149.10	20.17	14.14	54.12	28.95
408	159.30	55.15	46.90	6.46	56.26	44.91
409	75.10	66.32	4.25	4.25	22.53	22.53
410	151.65	145.15	20.17	14.14	54.12	28.95
411	159.30	55.15	46.90	6.46	56.26	44.91
412	251.01	229.64	27.16	27.16	75.30	75.30
413	159.30	116.35	32.39	6.46	56.26	44.91
414	159.30	116.35	33.92	6.46	56.26	44.91
415	159.30	75.13	21.38	6.46	56.26	44.91
416	159.30	75.13	21.38	6.46	56.26	44.91

## Design Ratio

Member ID	P	M <sub>z</sub>	M <sub>y</sub>	V <sub>y</sub>	V <sub>z</sub>	(P,M <sub>z</sub> ,M <sub>y</sub> )	Worst LC	KL/r	δ	Status
1	0.134	0.498	0.018	0.018	0.001	0.554	#13	0.745	Not Required	Pass
2	0.002	0.312	0.075	0.068	0.013	0.370	#21	0.116	Not Required	Pass
3	0.004	0.466	0.020	0.046	0.003	0.488	#21	0.046	Not Required	Pass
4	0.004	0.451	0.066	0.045	0.015	0.500	#21	0.082	Not Required	Pass
5	0.004	0.289	0.064	0.046	0.017	0.303	#21	0.076	Not Required	Pass
6	0.005	0.518	0.038	0.052	0.008	0.558	#21	0.046	Not Required	Pass
7	0.005	0.321	0.094	0.051	0.024	0.343	#21	0.076	Not Required	Pass
8	0.001	0.058	0.102	0.031	0.011	0.108	#24	0.102	Not Required	Pass

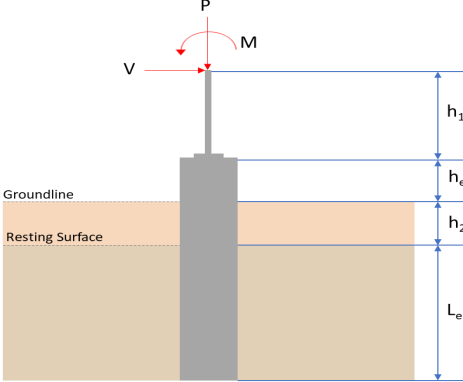
9	0.007	0.053	0.032	0.001	0.001	0.088	#21	0.206	Not Required	Pass
10	0.005	0.493	0.091	0.049	0.020	0.544	#21	0.082	Not Required	Pass
11	0.002	0.057	0.104	0.032	0.011	0.107	#24	0.102	Not Required	Pass
12	0.001	0.364	0.082	0.076	0.015	0.425	#21	0.054	Not Required	Pass
13	0.003	0.153	0.253	0.042	0.014	0.335	#21	0.306	Not Required	Pass
14	0.004	0.149	0.251	0.040	0.014	0.328	#24	0.204	Not Required	Pass
15	0.000	0.045	0.084	0.020	0.006	0.129	#21	Not Required	Not Required	Pass
16	0.000	0.044	0.084	0.020	0.006	0.128	#21	Not Required	Not Required	Pass
101	0.153	0.535	0.003	0.020	0.000	0.604	#13	0.745	Not Required	Pass
102	0.002	0.388	0.088	0.083	0.014	0.456	#21	0.116	Not Required	Pass
103	0.005	0.561	0.033	0.056	0.005	0.597	#21	0.046	Not Required	Pass
104	0.005	0.550	0.094	0.055	0.020	0.614	#21	0.082	Not Required	Pass
105	0.005	0.347	0.096	0.055	0.025	0.372	#21	0.076	Not Required	Pass
106	0.005	0.567	0.033	0.057	0.005	0.599	#21	0.046	Not Required	Pass
107	0.005	0.352	0.089	0.056	0.023	0.375	#21	0.076	Not Required	Pass
108	0.002	0.039	0.096	0.032	0.011	0.115	#21	0.102	Not Required	Pass
109	0.009	0.054	0.027	0.001	0.000	0.082	#21	0.206	Not Required	Pass
110	0.005	0.549	0.087	0.055	0.019	0.607	#21	0.082	Not Required	Pass
111	0.002	0.044	0.097	0.033	0.011	0.114	#21	0.102	Not Required	Pass
112	0.002	0.391	0.090	0.083	0.016	0.460	#21	0.036	Not Required	Pass
113	0.004	0.165	0.263	0.043	0.014	0.404	#21	0.306	Not Required	Pass
114	0.005	0.173	0.261	0.042	0.014	0.409	#21	0.306	Not Required	Pass
115	0.003	0.229	0.131	0.033	0.011	0.364	#21	0.507	Not Required	Pass
116	0.002	0.221	0.132	0.033	0.011	0.353	#21	0.507	Not Required	Pass
201	0.152	0.543	0.000	0.020	0.000	0.610	#13	0.745	Not Required	Pass
202	0.002	0.388	0.089	0.082	0.015	0.456	#21	0.036	Not Required	Pass
203	0.005	0.563	0.032	0.056	0.005	0.598	#21	0.046	Not Required	Pass
204	0.005	0.544	0.086	0.054	0.019	0.604	#21	0.082	Not Required	Pass
205	0.005	0.349	0.089	0.056	0.023	0.372	#21	0.076	Not Required	Pass
206	0.005	0.563	0.032	0.056	0.005	0.598	#21	0.046	Not Required	Pass
207	0.005	0.349	0.089	0.056	0.023	0.372	#21	0.076	Not Required	Pass
208	0.002	0.039	0.094	0.032	0.011	0.120	#21	0.102	Not Required	Pass
209	0.008	0.053	0.025	0.001	0.000	0.082	#21	0.206	Not Required	Pass
210	0.005	0.544	0.086	0.054	0.019	0.604	#21	0.082	Not Required	Pass
211	0.002	0.043	0.095	0.033	0.011	0.120	#21	0.102	Not Required	Pass
212	0.002	0.388	0.089	0.082	0.015	0.456	#21	0.036	Not Required	Pass
213	0.004	0.171	0.242	0.042	0.014	0.395	#21	0.306	Not Required	Pass
214	0.005	0.173	0.240	0.041	0.014	0.392	#21	0.306	Not Required	Pass
215	0.003	0.203	0.131	0.033	0.011	0.337	#21	0.507	Not Required	Pass
216	0.003	0.192	0.132	0.032	0.011	0.323	#21	0.507	Not Required	Pass
301	0.153	0.535	0.003	0.020	0.000	0.604	#13	0.745	Not Required	Pass
302	0.002	0.391	0.090	0.083	0.016	0.460	#21	0.036	Not Required	Pass
303	0.005	0.567	0.033	0.057	0.005	0.599	#21	0.046	Not Required	Pass
304	0.005	0.549	0.087	0.055	0.019	0.607	#21	0.082	Not Required	Pass
305	0.005	0.352	0.089	0.056	0.023	0.375	#21	0.076	Not Required	Pass
306	0.005	0.561	0.033	0.056	0.005	0.597	#21	0.046	Not Required	Pass
307	0.005	0.347	0.096	0.055	0.025	0.372	#21	0.076	Not Required	Pass
308	0.001	0.046	0.110	0.033	0.011	0.132	#21	0.102	Not Required	Pass
309	0.009	0.054	0.027	0.001	0.000	0.082	#21	0.206	Not Required	Pass
310	0.005	0.550	0.094	0.055	0.020	0.614	#21	0.082	Not Required	Pass
311	0.002	0.052	0.111	0.033	0.011	0.128	#21	0.102	Not Required	Pass
312	0.002	0.388	0.088	0.083	0.014	0.456	#21	0.116	Not Required	Pass
313	0.004	0.165	0.263	0.043	0.014	0.404	#21	0.306	Not Required	Pass
314	0.005	0.173	0.261	0.042	0.014	0.409	#21	0.306	Not Required	Pass

315	0.003	0.205	0.131	0.033	0.011	0.336	#21	0.507	Not Required	Pass
316	0.003	0.191	0.132	0.032	0.011	0.324	#21	0.507	Not Required	Pass
401	0.134	0.498	0.018	0.018	0.001	0.554	#13	0.745	Not Required	Pass
402	0.001	0.364	0.082	0.076	0.015	0.425	#21	0.054	Not Required	Pass
403	0.005	0.518	0.038	0.052	0.008	0.558	#21	0.046	Not Required	Pass
404	0.005	0.493	0.091	0.049	0.020	0.544	#21	0.082	Not Required	Pass
405	0.005	0.321	0.094	0.051	0.024	0.343	#21	0.076	Not Required	Pass
406	0.004	0.466	0.020	0.046	0.003	0.488	#21	0.046	Not Required	Pass
407	0.004	0.289	0.064	0.046	0.017	0.303	#21	0.076	Not Required	Pass
408	0.000	0.044	0.084	0.020	0.006	0.128	#21	Not Required	Not Required	Pass
409	0.007	0.053	0.032	0.001	0.001	0.088	#21	0.206	Not Required	Pass
410	0.004	0.451	0.066	0.045	0.015	0.500	#21	0.082	Not Required	Pass
411	0.000	0.045	0.084	0.020	0.006	0.129	#21	Not Required	Not Required	Pass
412	0.002	0.312	0.075	0.068	0.013	0.370	#21	0.116	Not Required	Pass
413	0.003	0.153	0.253	0.042	0.014	0.335	#21	0.204	Not Required	Pass
414	0.004	0.149	0.251	0.040	0.014	0.328	#24	0.306	Not Required	Pass
415	0.003	0.235	0.131	0.032	0.011	0.366	#21	0.507	Not Required	Pass
416	0.002	0.225	0.132	0.031	0.011	0.356	#21	0.507	Not Required	Pass

## Definitions

$\Phi_t$	Safety factor for tensile
$\Phi_c$	Safety factor for compression
$\Phi_b$	Safety factor for flexure
$\Phi_v$	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
$\delta$	Design ratio in case of member deflection
OK	Capacity is provided
NG	Capacity is not provided



REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6.5</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1192 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>7.566</td> <td>11.437</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.249</td> <td>-2.086</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.080</td> <td>0.125</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.452</td> <td>0.699</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>23.889</td> <td>41.447</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	7.566	11.437	$V_x$ (kip)	-1.249	-2.086	$V_z$ (kip)	0.080	0.125	$M_x$ (kipft)	0.452	0.699	$M_z$ (kipft)	23.889	41.447	
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	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.249 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.19889 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(23.889 \text{ kipft}) + ((-1.249 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.804 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1366 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.08 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.012739 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.452 \text{ kipft}) + ((0.08 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.071975 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.9342 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1366 \text{ ft}), (1.9342 \text{ ft})]$$

$$L_{e,req} = 6.137 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(6.137 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.94415$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.566 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47287 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47287 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23644$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.19889 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.804 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.804 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.804 \text{ kipft/ft})) + (4 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4334 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.804 \text{ kipft/ft})) + (3 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.804 \text{ kipft/ft})) + (2 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.25852 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (3.804 \text{ kipft/ft})) + ((-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.89683 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4334 \text{ ft})}{2}$$

$$p_a = 0.3325 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25852 \text{ kip/ft}^2)}{(0.3325 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7775$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

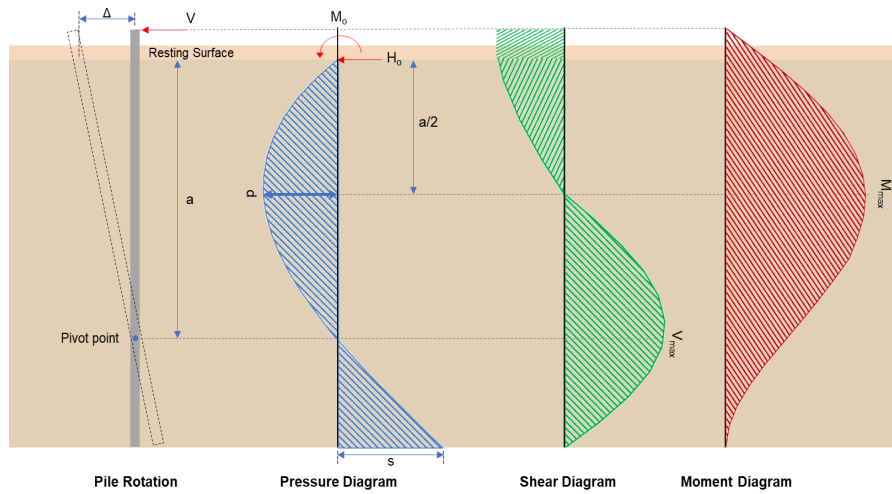
Status: **PASS**  
Ratio: **0.780**

	$p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89683 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.91983$	Status: <b>PASS</b> Ratio: <b>0.920</b>
	<p><b>Considering z-direction:</b></p> <p><math>H_o = 0.012739 \text{ kip/ft}</math> - Lateral force per length of pile,  <math>M_o = 0.071975 \text{ kipft/ft}</math> - Overturning moment per length of pile,  <math>a</math> - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.071975 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.012739 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.071975 \text{ kipft/ft})) + (4 \times (0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))}$ $a = 4.5684 \text{ ft}$ <p><math>p</math> - Earth pressure against the pile at distance <math>a/2</math> from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.071975 \text{ kipft/ft})) + (3 \times (0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (0.071975 \text{ kipft/ft})) + (2 \times (0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$ $p = 0.013382 \text{ kip/ft}^2$ <p><math>s</math> - Earth pressure against the pile at distance <math>L_e</math>,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.071975 \text{ kipft/ft})) + ((0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$ $s = 0.032201 \text{ kip/ft}^2$ <p><b>Check lateral soil pressure capacity:</b></p> <p><math>p_a</math> - Allowable lateral soil pressure at depth <math>a/2</math>,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.5684 \text{ ft})}{2}$ $p_a = 0.34263 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.013382 \text{ kip/ft}^2)}{(0.34263 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.039057$ <p><math>p_s</math> - Allowable lateral soil pressure at depth <math>L_e</math>,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$ $p_s = 0.975 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$	Status: <b>PASS</b> Ratio: <b>0.040</b>

$$Ratio = \frac{(0.032201 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.033027$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.086 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.33217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(41.447 \text{ kipft}) + ((-2.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5998 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5998 \text{ kipft/ft})}{(-0.33217 \text{ kip/ft})}$$

$$E = 19.869 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5998 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.5998 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4303 \text{ ft}$$

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

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

$$V_{max} = ((-0.33217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4303 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4303 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0702 \text{ kip}$$

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

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

$$M_{max} = ((-0.33217 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(19.869 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.439 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.125 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.019904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.699 \text{ kipft}) + ((0.125 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11131 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11131 \text{ kipft/ft})}{(0.019904 \text{ kip/ft})}$$

$$E = 5.592 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11131 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.019904 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.11131 \text{ kipft/ft})) + (4 \times (0.019904 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5698 \text{ ft}$$

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

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

$$V_{max} = ((0.019904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.5698 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.5698 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.17387 \text{ kip}$$

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

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

$$M_{max} = ((0.019904 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(5.592 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.51854 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(11.437 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.88 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.88 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.437 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0035927$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

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

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 11.437 \text{ kip} \rightarrow 11437 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(11437 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.32 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

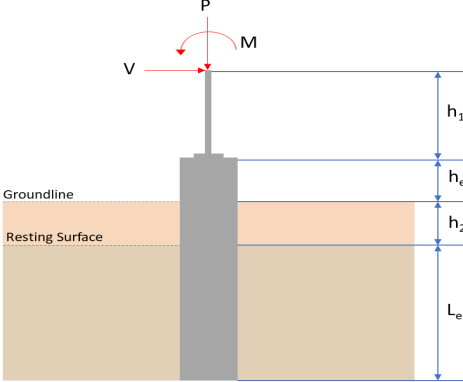
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.32 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.32 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.32 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.44 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.0702 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.0702 \text{ kip})}{(118.44 \text{ kip})}$ $\text{Ratio} = 0.068139$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.17387 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.17387 \text{ kip})}{(118.44 \text{ kip})}$ $\text{Ratio} = 0.001468$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 25.439\text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.439\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.093038$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.51854\text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.51854\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0018965$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6.5</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.627</td> <td>13.086</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.375</td> <td>-2.291</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.013</td> <td>-0.024</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.073</td> <td>-0.133</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>25.717</td> <td>44.575</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	8.627	13.086	$V_x$ (kip)	-1.375	-2.291	$V_z$ (kip)	-0.013	-0.024	$M_x$ (kipft)	-0.073	-0.133	$M_z$ (kipft)	25.717	44.575	
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$M_z$ (kipft)	25.717	44.575																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.375 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.21895 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.717 \text{ kipft}) + ((-1.375 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0951 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.2604 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0020701 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.073 \text{ kipft}) + ((-0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011624 \text{ kipft/ft}$$

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:

$L_{e,z} = 0.93357 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.2604 \text{ ft}), (0.93357 \text{ ft})]$$

$$L_{e,req} = 6.26 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(6.26 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.96308$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.627 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53919 \text{ kip/ft}^2$$

$$q = 0.53919 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26959$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.21895 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0951 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0951 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.0951 \text{ kipft/ft})) + (4 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4352 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (4.0951 \text{ kipft/ft})) + (3 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (4.0951 \text{ kipft/ft})) + (2 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.27584 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (4.0951 \text{ kipft/ft})) + ((-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.961 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4352 \text{ ft})}{2}$$

$$p_a = 0.33264 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27584 \text{ kip/ft}^2)}{(0.33264 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82924$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.961 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98563$$

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

**Considering z-direction:**

$H_o = -0.0020701 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.011624 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011624 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.011624 \text{ kipft/ft})) + (4 \times (-0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5693 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.011624 \text{ kipft/ft})) + (3 \times (-0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.011624 \text{ kipft/ft})) + (2 \times (-0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.000083797 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (0.011624 \text{ kipft/ft})) + ((-0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0013907 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5693 \text{ ft})}{2}$$

$$p_a = 0.34269 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.000083797 \text{ kip/ft}^2)}{(0.34269 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00024452$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

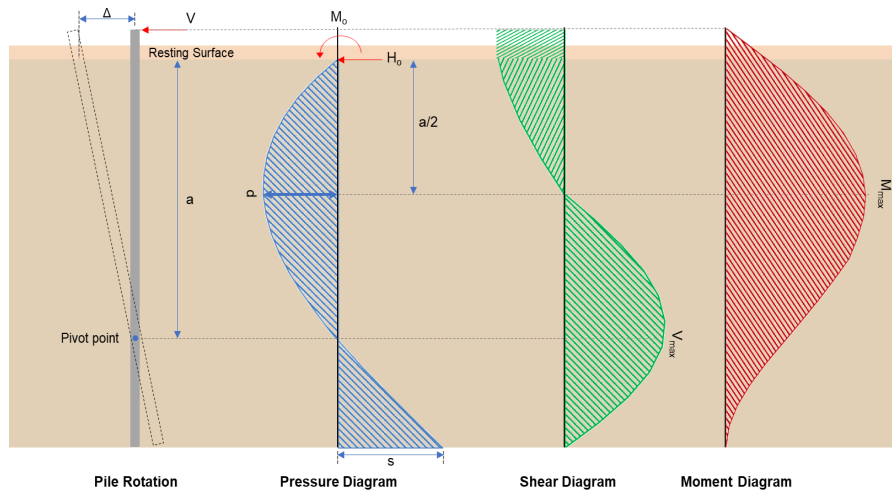
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0013907 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.0014264$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.291 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36481 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(44.575 \text{ kipft}) + ((-2.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0979 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.0979 \text{ kipft/ft})}{(-0.36481 \text{ kip/ft})}$$

$$E = 19.457 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0979 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.36481 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.0979 \text{ kipft/ft})) + (4 \times (-0.36481 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.432 \text{ ft}$$

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

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

$$V_{max} = ((-0.36481 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.432 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.432 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6989 \text{ kip}$$

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

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

$$M_{max} = ((-0.36481 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(19.457 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 27.405 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.133 \text{ kipft}) + ((-0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.021178 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.021178 \text{ kipft/ft})}{(-0.0038217 \text{ kip/ft})}$$

$$E = 5.5417 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021178 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.021178 \text{ kipft/ft})) + (4 \times (-0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.571 \text{ ft}$$

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

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

$$V_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.571 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.571 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.033174 \text{ kip}$$

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

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

$$M_{max} = ((-0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(5.5417 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.098884 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.086 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.83 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.83 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

25.7.2.2

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

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.086 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0041107$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 13.086 \text{ kip} \rightarrow 13086 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(13086 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.54 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

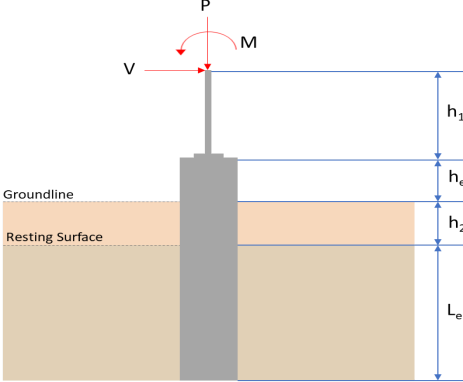
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.54 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.54 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.58 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.6989 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.6989 \text{ kip})}{(118.58 \text{ kip})}$ $\text{Ratio} = 0.073358$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.033174 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.033174 \text{ kip})}{(118.58 \text{ kip})}$ $\text{Ratio} = 0.00027976$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),          Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 27.405\text{kipft}</math> - Maximum moment in the x-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(27.405\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.10023$	<p>Status: <b>PASS</b>          Ratio: <b>0.100</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.098884\text{kipft}</math> - Maximum moment in the z-direction,          Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.098884\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00036165$	<p>Status: <b>PASS</b>          Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b></p> <p>Pile shape: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 6.5</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.627</td> <td>13.086</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.375</td> <td>-2.291</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.013</td> <td>0.024</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.073</td> <td>0.133</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>25.717</td> <td>44.576</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	8.627	13.086	$V_x$ (kip)	-1.375	-2.291	$V_z$ (kip)	0.013	0.024	$M_x$ (kipft)	0.073	0.133	$M_z$ (kipft)	25.717	44.576	
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	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><math>H</math> - 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><b>Considering x-direction:</b></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.375 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.21895 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(25.717 \text{ kipft}) + ((-1.375 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.0951 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.2604 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.013 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0020701 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.073 \text{ kipft}) + ((0.013 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.011624 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.0184 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.2604 \text{ ft}), (1.0184 \text{ ft})]$$

$$L_{e,req} = 6.26 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(6.26 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.96308$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.627 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53919 \text{ kip/ft}^2$$

$$q = 0.53919 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.53919 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26959$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.21895 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 4.0951 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.0951 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (4.0951 \text{ kipft/ft})) + (4 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4352 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (4.0951 \text{ kipft/ft})) + (3 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 [(3 \times (4.0951 \text{ kipft/ft})) + (2 \times (-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.27584 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 [(2 \times (4.0951 \text{ kipft/ft})) + ((-0.21895 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.961 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4352 \text{ ft})}{2}$$

$$p_a = 0.33264 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27584 \text{ kip/ft}^2)}{(0.33264 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.82924$$

$p_a$  - Allowable lateral soil pressure at depth  $L_e$ ,

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

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.961 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.98563$$

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

**Considering z-direction:**

$H_o = 0.0020701 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.011624 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.011624 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.011624 \text{ kipft/ft})) + (4 \times (0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5693 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.011624 \text{ kipft/ft})) + (3 \times (0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.011624 \text{ kipft/ft})) + (2 \times (0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.0021679 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (0.011624 \text{ kipft/ft})) + ((0.0020701 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0052124 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5693 \text{ ft})}{2}$$

$$p_a = 0.34269 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0021679 \text{ kip/ft}^2)}{(0.34269 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0063259$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

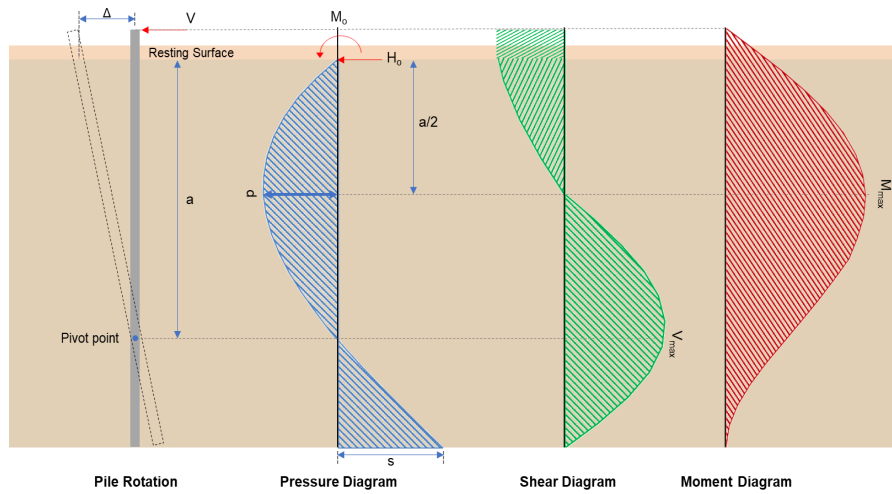
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0052124 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.005346$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.291 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36481 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(44.576 \text{ kipft}) + ((-2.291 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.0981 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.0981 \text{ kipft/ft})}{(-0.36481 \text{ kip/ft})}$$

$$E = 19.457 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.0981 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.36481 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (7.0981 \text{ kipft/ft})) + (4 \times (-0.36481 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.432 \text{ ft}$$

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

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

$$V_{max} = ((-0.36481 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.432 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.432 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.6991 \text{ kip}$$

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

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

$$M_{max} = ((-0.36481 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(19.457 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (19.457 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.432 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.405 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(0.024 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0038217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.133 \text{ kipft}) + ((0.024 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.021178 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.021178 \text{ kipft/ft})}{(0.0038217 \text{ kip/ft})}$$

$$E = 5.5417 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.021178 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.021178 \text{ kipft/ft})) + (4 \times (0.0038217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.571 \text{ ft}$$

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

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

$$V_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.571 \text{ ft})}{(6.5 \text{ ft})} \right)^2 + 4 \times \left( \frac{3 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.571 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.033174 \text{ kip}$$

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

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

$$M_{max} = ((0.0038217 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(5.5417 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 + \left[ \left( \frac{3 \times (5.5417 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.571 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.098884 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.086 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.83 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.83 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

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

25.7.2.1  $s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.086 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0041107$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

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

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 13.086 \text{ kip} \rightarrow 13086 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(13086 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.54 \text{ kip}$$

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

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

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

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

$V_c$  - Governing shear strength of concrete

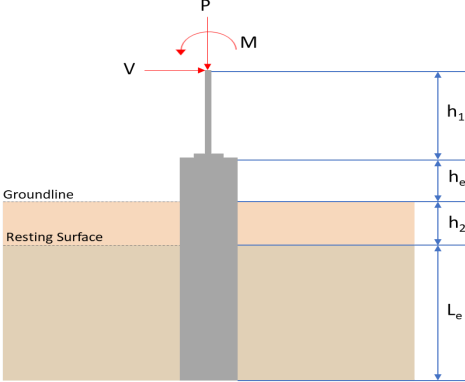
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.54 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.54 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.54 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.58 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.6991 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.6991 \text{ kip})}{(118.58 \text{ kip})}$ $\text{Ratio} = 0.07336$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.033174 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.033174 \text{ kip})}{(118.58 \text{ kip})}$ $\text{Ratio} = 0.00027976$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 27.405\text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(27.405\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.10023$	<p>Status: <b>PASS</b>  Ratio: <b>0.100</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.098884\text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.098884\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.00036165$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b></p> <p>Pile shape: rectangular  <math>b = 48</math> in - Pile width  <math>D = 48</math> in - Pile depth  <math>L = 6.5</math> ft - Total pile length  <math>h_1 = 0</math> ft - Lateral load height from the top of the pile,  <math>h_2 = 0</math> ft - Depth to resting surface  <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>7.566</td> <td>11.437</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.249</td> <td>-2.086</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>-0.080</td> <td>-0.125</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>-0.452</td> <td>-0.699</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>23.889</td> <td>41.447</td> </tr> </tbody> </table> <p><b>Material Properties</b></p> <p><math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	7.566	11.437	$V_x$ (kip)	-1.249	-2.086	$V_z$ (kip)	-0.080	-0.125	$M_x$ (kipft)	-0.452	-0.699	$M_z$ (kipft)	23.889	41.447	
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	<p><b>Required depth to resist lateral loads (ASD)</b></p> <p><math>H</math> - 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><b>Considering x-direction:</b></p> <p><math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.249 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.19889 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(23.889 \text{ kipft}) + ((-1.249 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.804 \text{ kipft/ft}$$

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:

$L_{e,x} = 6.1366 \text{ ft}$  - Required depth in x-direction,

**Considering z-direction:**

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.08 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.012739 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.452 \text{ kipft}) + ((-0.08 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.071975 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.6505 \text{ ft}$  - Required depth in z-direction,

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[(6.1366 \text{ ft}), (1.6505 \text{ ft})]$$

$$L_{e,req} = 6.137 \text{ ft}$$

$L_e$  - 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

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

$$\text{Ratio} = \frac{(6.137 \text{ ft})}{(6.5 \text{ ft})}$$

$$\text{Ratio} = 0.94415$$

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

**End-bearing Capacity (ASD)**

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.566 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47287 \text{ kip/ft}$$

**Check bearing capacity ratio:**

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47287 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23644$$

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

Czerniak

**Lateral Soil Pressure (ASD):**

$L/D$  - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.625$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.19889 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 3.804 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.804 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (3.804 \text{ kipft/ft})) + (4 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4334 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.804 \text{ kipft/ft})) + (3 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (3.804 \text{ kipft/ft})) + (2 \times (-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.25852 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (3.804 \text{ kipft/ft})) + ((-0.19889 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.89683 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.4334 \text{ ft})}{2}$$

$$p_a = 0.3325 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25852 \text{ kip/ft}^2)}{(0.3325 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.7775$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

Status: **PASS**  
Ratio: **0.780**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.89683 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.91983$$

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

**Considering z-direction:**

$H_o = -0.012739 \text{ kip/ft}$  - Lateral force per length of pile,

$M_o = 0.071975 \text{ kipft/ft}$  - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.071975 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.012739 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.071975 \text{ kipft/ft})) + (4 \times (-0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5684 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.071975 \text{ kipft/ft})) + (3 \times (-0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]^2}{(6.5 \text{ ft})^2 \times [(3 \times (0.071975 \text{ kipft/ft})) + (2 \times (-0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]}$$

$$p = 0.00055016 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (0.071975 \text{ kipft/ft})) + ((-0.012739 \text{ kip/ft}) \times (6.5 \text{ ft}))]}{(6.5 \text{ ft})^2}$$

$$s = 0.0086835 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5684 \text{ ft})}{2}$$

$$p_a = 0.34263 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.00055016 \text{ kip/ft}^2)}{(0.34263 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0016057$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.5 \text{ ft})$$

$$p_s = 0.975 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

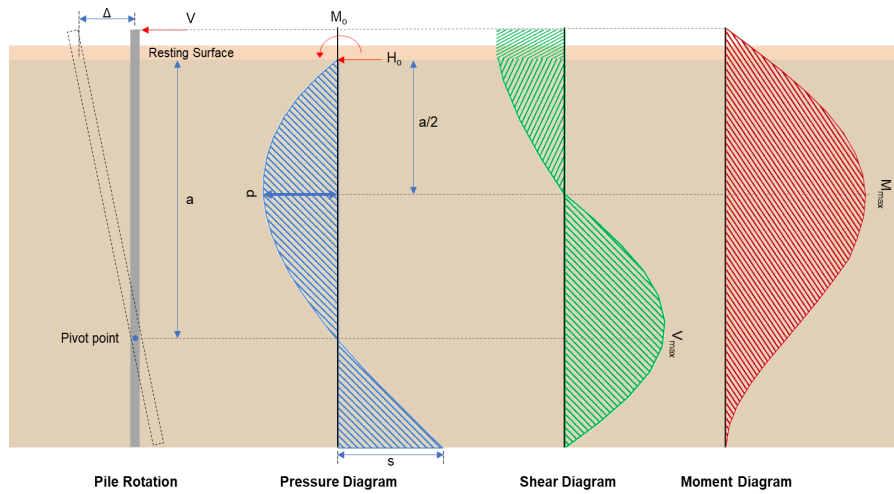
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.0086835 \text{ kip/ft}^2)}{(0.975 \text{ kip/ft}^2)}$$

$$Ratio = 0.0089062$$

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



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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.086 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.33217 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(41.447 \text{ kipft}) + ((-2.086 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.5998 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.5998 \text{ kipft/ft})}{(-0.33217 \text{ kip/ft})}$$

$$E = 19.869 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.5998 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.33217 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (6.5998 \text{ kipft/ft})) + (4 \times (-0.33217 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.4303 \text{ ft}$$

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

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

$$V_{max} = ((-0.33217 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4303 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4303 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.0702 \text{ kip}$$

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

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

$$M_{max} = ((-0.33217 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(19.869 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (19.869 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.4303 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 25.439 \text{ kipft}$$

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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-0.125 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.019904 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

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

$$M_o = \frac{(0.699 \text{ kipft}) + ((-0.125 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11131 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.11131 \text{ kipft/ft})}{(-0.019904 \text{ kip/ft})}$$

$$E = 5.592 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11131 \text{ kipft/ft}) \times (6.5 \text{ ft})) + (3 \times (-0.019904 \text{ kip/ft}) \times (6.5 \text{ ft})^2)}{(6 \times (0.11131 \text{ kipft/ft})) + (4 \times (-0.019904 \text{ kip/ft}) \times (6.5 \text{ ft}))}$$

$$a = 4.5698 \text{ ft}$$

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

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

$$V_{max} = ((-0.019904 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.5698 \text{ ft})}{(6.5 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.5698 \text{ ft})}{(6.5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.17387 \text{ kip}$$

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

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

$$M_{max} = ((-0.019904 \text{ kip/ft}) \times (48 \text{ in}) \times (6.5 \text{ ft})) \times \left[ \left( \frac{(5.592 \text{ ft})}{(6.5 \text{ ft})} + \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right) - \left[ \left( \frac{4 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 3 \right) \times \left( \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (5.592 \text{ ft})}{(6.5 \text{ ft})} + 2 \right) \times \left( \frac{(4.5698 \text{ ft})}{2 \times (6.5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.51854 \text{ kipft}$$

**Minimum Reinforcement Check (LRFD)**

**Parameters:**

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

Table 22.4.2.1

**Longitudinal reinforcement:**

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(11.437 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.88 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.88 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

**Ties:**

25.7.2.2

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

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

**Summary:**

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

Ties: #3(0.375 in) - 10 in

**Axial Compression Strength (ACI 318-19, LRFD)**

22.4.2.2

$\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.437 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0035927$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

$\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 11.437 \text{ kip} \rightarrow 11437 \text{ lbf}$ ,  
 $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(11437 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.32 \text{ kip}$$

22.5.5.1.2

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

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

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

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

$V_c$  - Governing shear strength of concrete

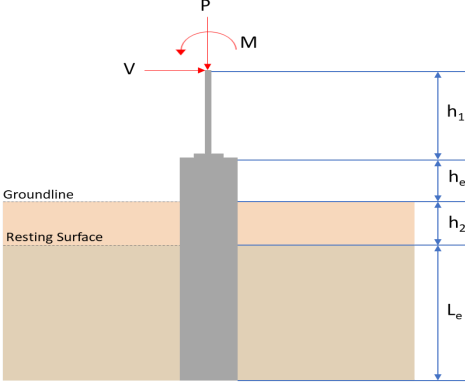
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.32 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.32 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>,  <math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.32 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.44 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.0702 \text{ kip}</math> - Maximum shear force in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.0702 \text{ kip})}{(118.44 \text{ kip})}$ $\text{Ratio} = 0.068139$ <p><b>Considering z-direction:</b></p> <p><math>V_{max} = 0.17387 \text{ kip}</math> - Maximum shear force in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.17387 \text{ kip})}{(118.44 \text{ kip})}$ $\text{Ratio} = 0.001468$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p> <p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>
	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$	

<p>14.5.2.1b</p>	<p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:  <math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3\text{ksi}} \times 18432.001\text{in}^3$ $\phi M_{n,1} = 273.423\text{kipft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3\text{ksi}) \times (18432\text{in}^3)$ $\phi M_{n,2} = 2545.9\text{kipft}$ <p>Therefore,  <math>\phi M_n</math> - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42\text{kipft}), (2545.9\text{kipft})]$ $\phi M_n = 273.42\text{kipft}$ <p><b>Considering x-direction:</b>  <math>M_{max} = 25.439\text{kipft}</math> - Maximum moment in the x-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(25.439\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.093038$	<p>Status: <b>PASS</b>  Ratio: <b>0.090</b></p>
	<p><b>Considering z-direction:</b>  <math>M_{max} = 0.51854\text{kipft}</math> - Maximum moment in the z-direction,  Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.51854\text{kipft})}{(273.42\text{kipft})}$ $\text{Ratio} = 0.0018965$	<p>Status: <b>PASS</b>  Ratio: <b>0.000</b></p>

REFERENCES	CALCULATIONS	RESULTS																										
	<p><b>SkyCiv Foundation Design</b> Pile Foundation</p> <p><b>Design Information :</b> Design code : IBC 2021 (International Building Code) Unit System : Imperial</p>																											
	<p><b>Pile Input</b></p>  <p><b>Geometry</b> Pile shape: rectangular <math>b = 48</math> in - Pile width <math>D = 48</math> in - Pile depth <math>L = 6.75</math> ft - Total pile length <math>h_1 = 0</math> ft - Lateral load height from the top of the pile, <math>h_2 = 0</math> ft - Depth to resting surface <math>h_e = 0</math> ft - Length of pile above the ground</p> <p><b>Tabulation of Soil Parameters</b></p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (<math>q_a</math>) (psf)</th> <th>Allowable Lateral Pressure (<math>R</math>) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel &amp; clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p><b>Tabulation of Loads</b></p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td><math>P</math> (kip)</td> <td>8.585</td> <td>13.014</td> </tr> <tr> <td><math>V_x</math> (kip)</td> <td>-1.387</td> <td>-2.310</td> </tr> <tr> <td><math>V_z</math> (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_x</math> (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td><math>M_z</math> (kipft)</td> <td>26.089</td> <td>45.230</td> </tr> </tbody> </table> <p><b>Material Properties</b> <math>f'_{ck} = 3</math> ksi - Concrete strength,</p>	Layer	Label	Allowable Bearing Pressure ( $q_a$ ) (psf)	Allowable Lateral Pressure ( $R$ ) (psf/ft)	1	Sand, silty sand, clayey sand, silty gravel & clayey gravel	2000.000	150.000	Load Component	ASD	LRFD	$P$ (kip)	8.585	13.014	$V_x$ (kip)	-1.387	-2.310	$V_z$ (kip)	0.000	0.000	$M_x$ (kipft)	0.000	0.000	$M_z$ (kipft)	26.089	45.230	
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$M_x$ (kipft)	0.000	0.000																										
$M_z$ (kipft)	26.089	45.230																										
	<p><b>Required depth to resist lateral loads (ASD)</b> <math>H</math> - 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><b>Considering x-direction:</b> <math>H_o</math> - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.387 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.22086 \text{ kip/ft}$ <p><math>M_o</math> - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$																											

$$M_o = \frac{(26.089 \text{ kipft}) + ((-1.387 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.1543 \text{ kipft/ft}$$

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:

$$L_{e,x} = 6.291 \text{ ft} - \text{Required depth in x-direction,}$$

**Considering z-direction:**

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

**Minimum embedded depth required:**

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

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

$$L_{e,req} = \text{MAX}[6.291 \text{ ft}, (0 \text{ ft})]$$

$$L_{e,req} = 6.291 \text{ ft}$$

$L_e$  - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6.75 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 6.75 \text{ ft}$$

**Ratio** - Embedded depth

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

$$\text{Ratio} = \frac{(6.291 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.932$$

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

### End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.585 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.53656 \text{ kip/ft}^2$$

**Check bearing capacity ratio:**

**Ratio** - Capacity

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

$$\text{Ratio} = \frac{(0.53656 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.26828$$

Status: **PASS**  
Ratio: **0.270**

Czerniak

### Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.6875$$

Since  $L/D \leq 10$ ,

Pile is short.

**Considering x-direction:**

$H_o = -0.22086$  kip/ft - Lateral force per length of pile,

$M_o = 4.1543$  kipft/ft - Overturning moment per length of pile,

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.1543 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.22086 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.1543 \text{ kipft/ft})) + (4 \times (-0.22086 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6086 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (4.1543 \text{ kipft/ft})) + (3 \times (-0.22086 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (4.1543 \text{ kipft/ft})) + (2 \times (-0.22086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.25607 \text{ kip/ft}^2$$

$s$  - Earth pressure against the pile at distance  $L_e$ ,

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

$$s = \frac{6 \times [(2 \times (4.1543 \text{ kipft/ft})) + ((-0.22086 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.89782 \text{ kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.6086 \text{ ft})}{2}$$

$$p_a = 0.34564 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.25607 \text{ kip/ft}^2)}{(0.34564 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.74086$$

$p_s$  - Allowable lateral soil pressure at depth  $L_e$ ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

*Ratio* - Lateral soil capacity

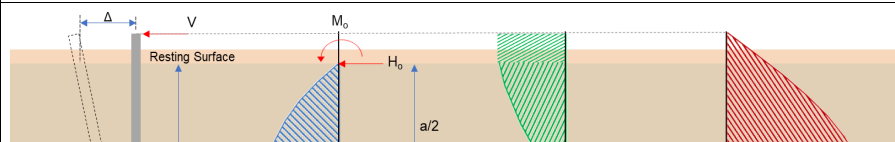
$$\text{Ratio} = \frac{s}{p_s}$$

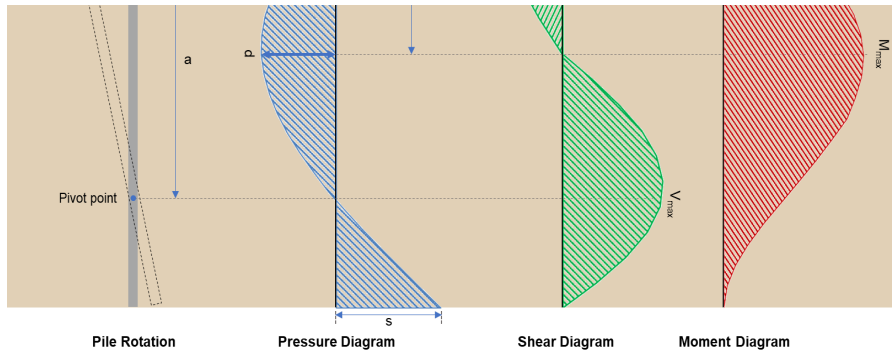
$$\text{Ratio} = \frac{(0.89782 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.88673$$

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

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





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

$H_o$  - Lateral force per length of pile,

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

$$H_o = \frac{(-2.31 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.36783 \text{ kip/ft}$$

$M_o$  - Moment per length of pile,

$$M_o = \frac{M_e + (V_e H)}{1.57 D}$$

$$M_o = \frac{(45.23 \text{ kipft}) + ((-2.31 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 7.2022 \text{ kipft/ft}$$

$E$  - Distance from lateral load to resisting surface,

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

$$E = \frac{(7.2022 \text{ kipft/ft})}{(-0.36783 \text{ kip/ft})}$$

$$E = 19.58 \text{ ft}$$

$a$  - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (7.2022 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.36783 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (7.2022 \text{ kipft/ft})) + (4 \times (-0.36783 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.6051 \text{ ft}$$

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

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

$$V_{max} = ((-0.36783 \text{ kip/ft}) \times (48 \text{ in})) \times \left[ 1 - \left[ 3 \times \left( \frac{4 \times (19.58 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6051 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[ 4 \times \left( \frac{3 \times (19.58 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6051 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.5293 \text{ kip}$$

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

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

$$M_{max} = ((-0.36783 \text{ kip/ft}) \times (48 \text{ in}) \times (6.75 \text{ ft})) \times \left[ \left( \frac{(19.58 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.6051 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[ \left( \frac{4 \times (19.58 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left( \frac{(4.6051 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[ \left( \frac{3 \times (19.58 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left( \frac{(4.6051 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.879 \text{ kipft}$$

### Minimum Reinforcement Check (LRFD)

#### Parameters:

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

Table 22.4.2.1

#### Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[ \frac{\frac{(13.014 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -101.83 \text{ in}^2$$

$A_{min}$  - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-101.83 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

$n_{rebar}$  - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$  - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

$s_{rebar}$  - Minimum spacing of reinforcement,

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

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

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

#### Ties:

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

25.7.2.1

$s_{ties}$  - Maximum spacing of ties,

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

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

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

#### Summary:

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

**Axial Compression Strength (ACI 318-19, LRFD)**22.4.2.2  $\phi P_N$  - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(13.014 \text{ kip})}{(3183.4 \text{ kip})}$$

$$\text{Ratio} = 0.0040881$$

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

$$d = 0.80 D$$

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

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

22.5.5.1.3  $\lambda_s$  - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

The following variables were converted to be consistent with empirical formula  $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$ ,  $P = 13.014 \text{ kip} \rightarrow 13014 \text{ lbf}$ ,22.5.5.1.1(a)  $V_{c,a}$  - Shear strength of concrete (a)

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

$$V_{c,a} = \left[ 2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(13014 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.53 \text{ kip}$$

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

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

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

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

 $V_c$  - Governing shear strength of concrete

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

$$V_c = \text{Min}[(324.49 \text{ kip}), (131.53 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.53 \text{ kip}$$

<p>22.5.1.2</p>	<p>The following variables were converted to be consistent with empirical formula <math>f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}</math>.</p> <p><math>V_{s,a}</math> - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 807.65 \text{ kip}$ <p><math>A_v</math> - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 <math>V_{s,b}</math> - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p><math>V_s</math> - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 <math>\phi V_n</math> - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.53 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.58 \text{ kip}$ <p><b>Considering x-direction:</b></p> <p><math>V_{max} = 8.5293 \text{ kip}</math> - Maximum shear force in the x-direction,  <b>Ratio</b> - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(8.5293 \text{ kip})}{(118.58 \text{ kip})}$ $Ratio = 0.071932$	<p>Status: <b>PASS</b>  Ratio: <b>0.070</b></p>
<p>14.5.2.1b</p>	<p><b>Flexural Strength (ACI 318-19, LRFD)</b></p> <p><math>S_m</math> - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p><math>\lambda = 1</math> - Concrete modification factor (Normal concrete),  Allowable flexural strength:  <math>M_n</math> shall be the lesser of:</p> <p><math>\phi M_{n,1}</math></p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kip ft}$ <p><math>\phi M_{n,2}</math></p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$	

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

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

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

Therefore,

$\phi M_n$  - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 27.879 \text{ kipft}$  - Maximum moment in the x-direction,

*Ratio* - Capacity

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

$$\text{Ratio} = \frac{(27.879 \text{ kipft})}{(273.42 \text{ kipft})}$$

$$\text{Ratio} = 0.10196$$

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