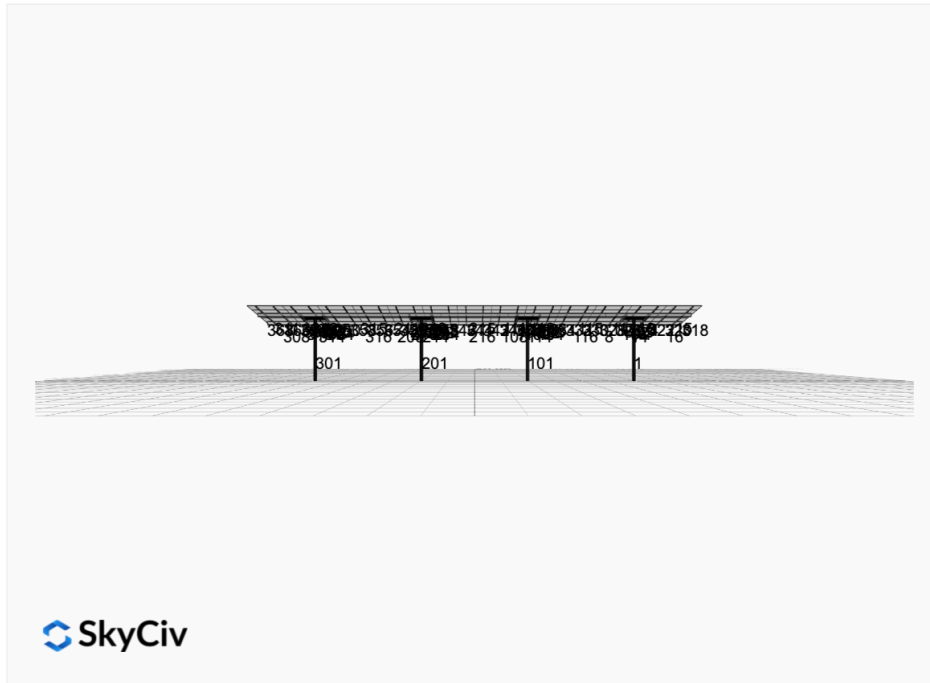


Project Name: Carport-Day-5x13 **Date:** Wed Feb 05 2025
Location: Joshua Tree, CA, USA **Number of Modules:** 65
Unique ID: 4P-19.75-6TOP-HD-72-L-5Hx13W-JKL5 **Number of Poles:** 4
Dealer: _____ **Date Sold:** _____



| | |
|-----------------------------|----------|
| Array Dimensions N/S | 17.29 ft |
| Array Dimensions E/W | 80.17 ft |
| Winter Tilt Angle | 10 |
| Front Edge Clearance | 10 ft |

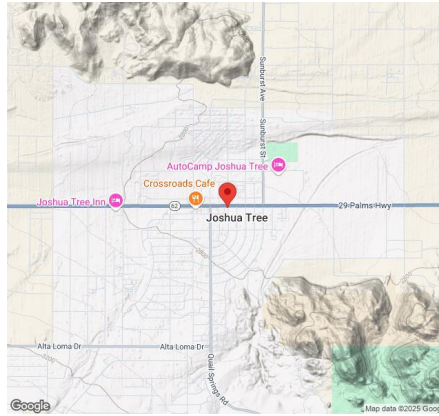
MT Solar Bill of Materials (4P-19.75-6TOP-HD-72-L-5Hx13W-JKL5)

| Part | Short Description | BOM Qty |
|---------------------|-----------------------|---------|
| MTS-PC-6 | 6IN Pole Cap Assembly | 4 |
| MTS-HF-HD | H-Frame Assembly-HD | 4 |
| MTS-HD-Wing-72 | 72IN HD Wing | 4 |
| MTS-HD-Splice-90 | 90IN HD Splice | 6 |
| MTS-HD-Splice-57 | 57IN HD Splice | 6 |
| MTS-CLAMP-ANGLE-4PK | Angle Clamp | 13 |

Rail Bill of Materials

| Part | Qty |
|------------------|-----|
| Rails (205in) | 26 |
| Rail Attachment | 104 |
| Module Mid Clamp | 104 |
| Module End Clamp | 52 |
| Ground Lug | 13 |

Site Details:



Site Address: Joshua Tree, CA, USA

Array Specification

| | |
|------------------------------------|-----------|
| Duty Classification: | HD |
| Module Width: | 41.00 in |
| Module Length: | 73.00in |
| Number of Rows: | 5 |
| Number of Columns: | 13 |
| Total Number of Modules: | 65 |
| Winter Tilt Angle: | 10 |
| Front Edge Clearance: | 10 |
| Total Array Height at Tilt: | 13.00 ft |
| Total Frame Length: | 78.75 ft |
| Frame Weight: | 4573 lbs |
| Array Dimensions N/S: | 17.29 ft |
| Array Dimensions E/W: | 80.17 ft |
| Rail Length: | 207.50 in |
| Rail Spacing: | 3.08 ft |

Support Specifications

| | |
|---------------------------------|-----------------|
| Pole Size: | 6in Pipe Sch 40 |
| Pole Length above Grade: | 11.50 ft |
| Number of Poles: | 4 |
| Pole Spacing: | 19.75 ft |

Foundation Specifications

| | |
|--|--|
| Foundation Type: | Square |
| Foundation Dimensions: | 48 x 48 in |
| Foundation Depth (below grade): | Pile 1: 6.00 ft Pile 2: 6.00 ft Pile 3: 6.00 ft Pile 4: 6.00 ft |
| Foundation Volume: | 14.222 y ³ |

Site Info

| | |
|-----------------------------|----------------------|
| Risk Category: | I |
| Exposure: | C |
| Soil Classification: | sand |
| Site Location: | Joshua Tree, CA, USA |
| Wind Speed: | 120 mph |

Snow Load:

5 psf

Design Disclaimer

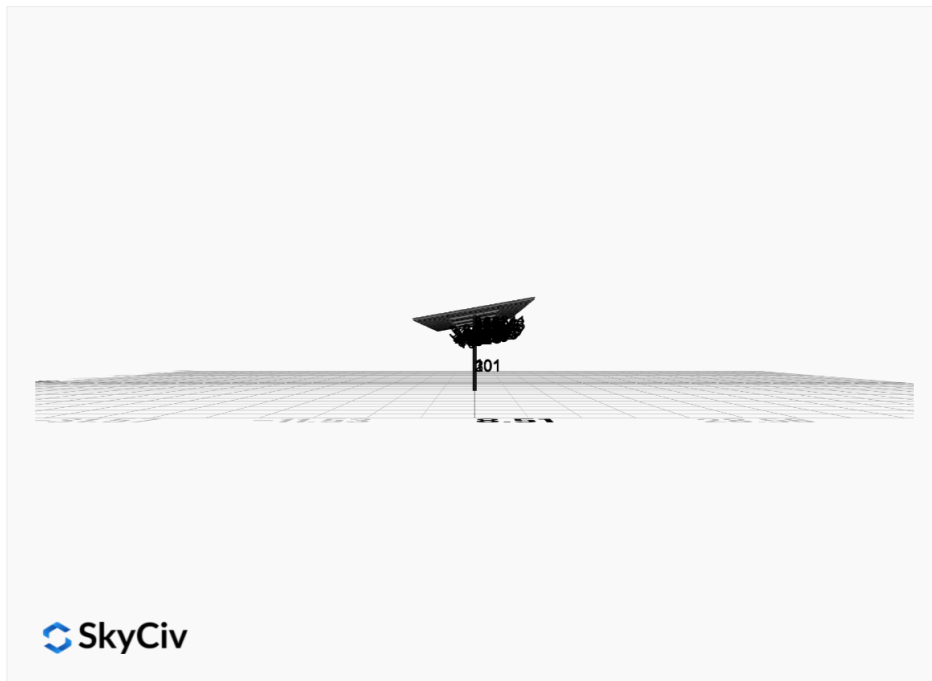
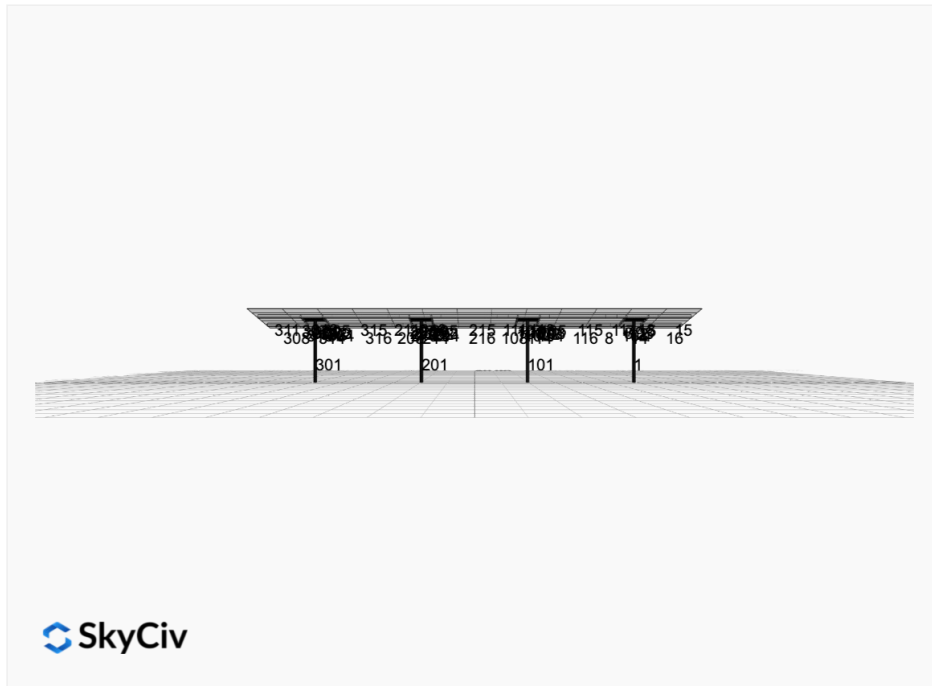
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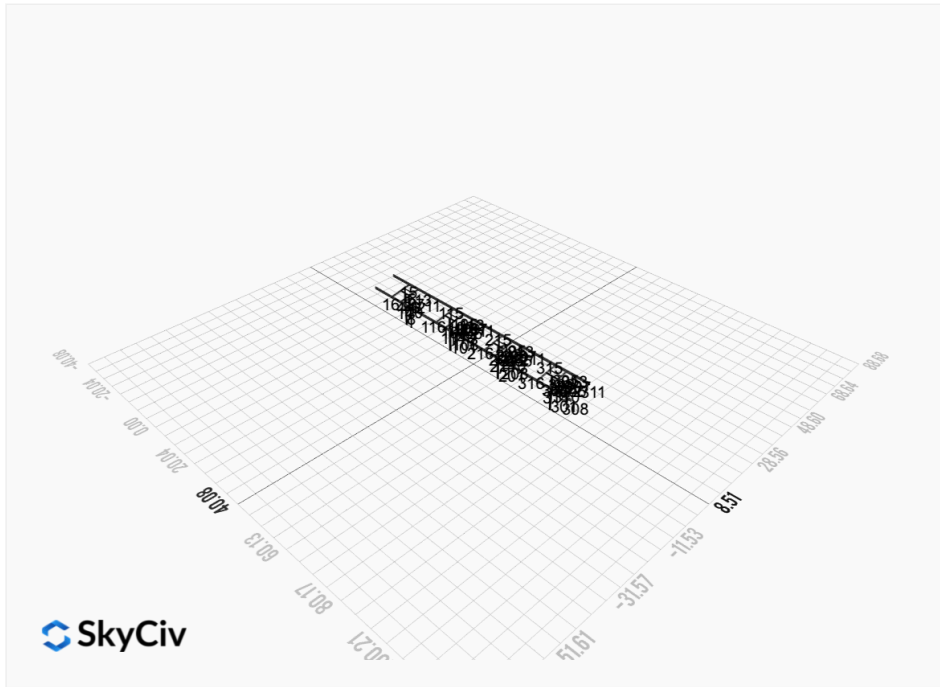
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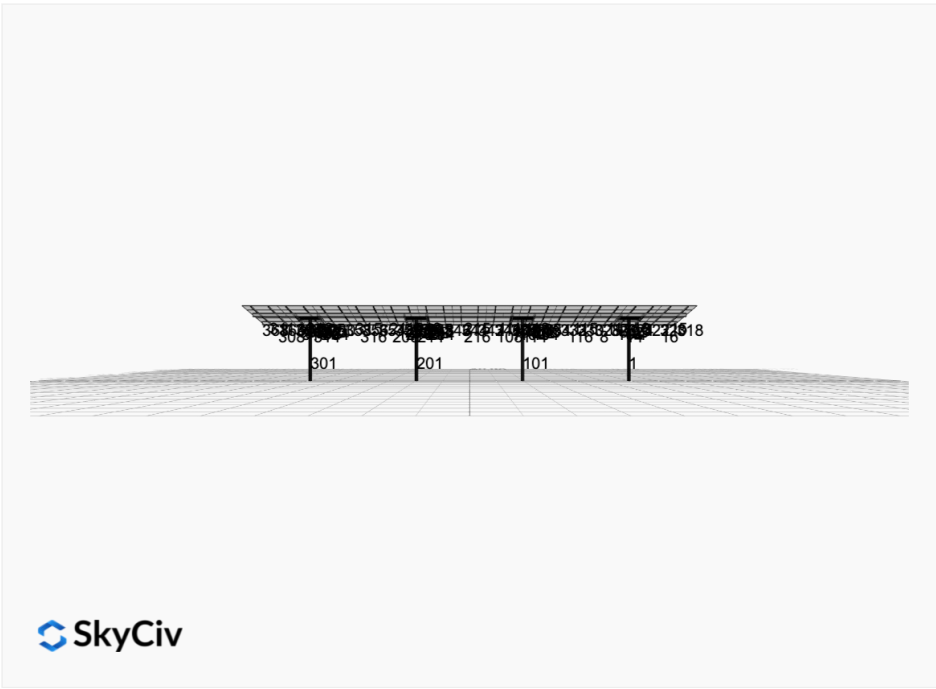
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Design Notes:

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

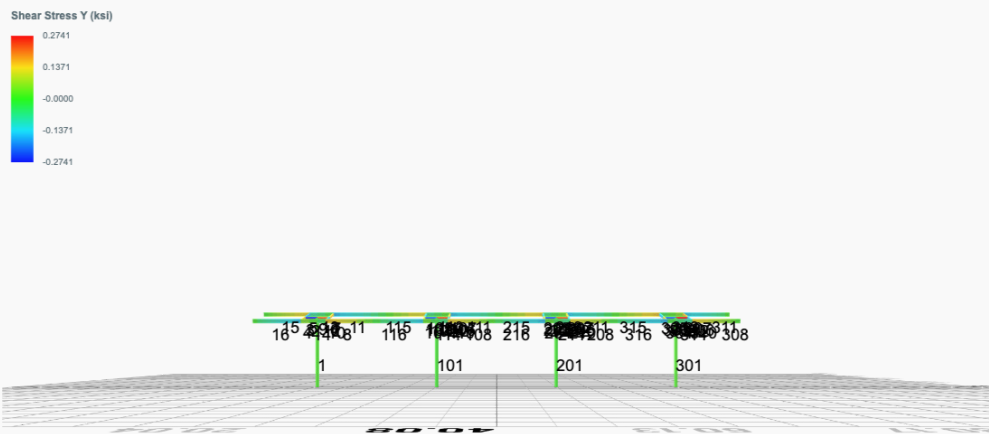
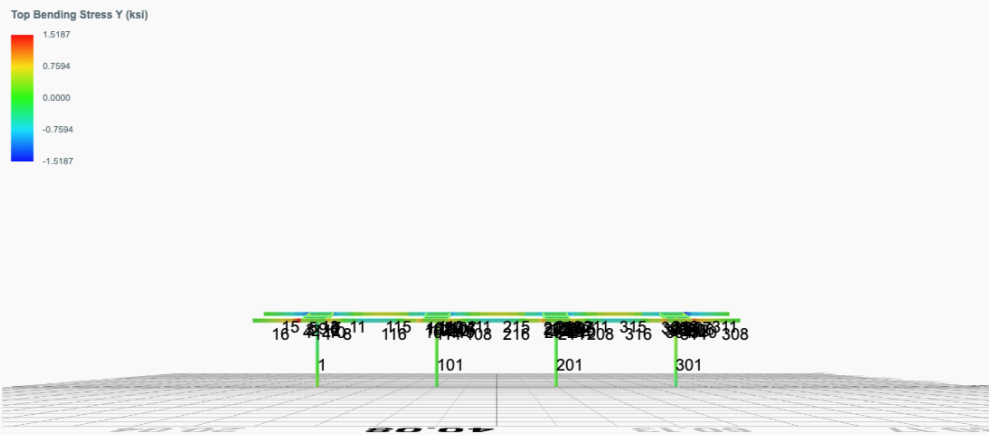


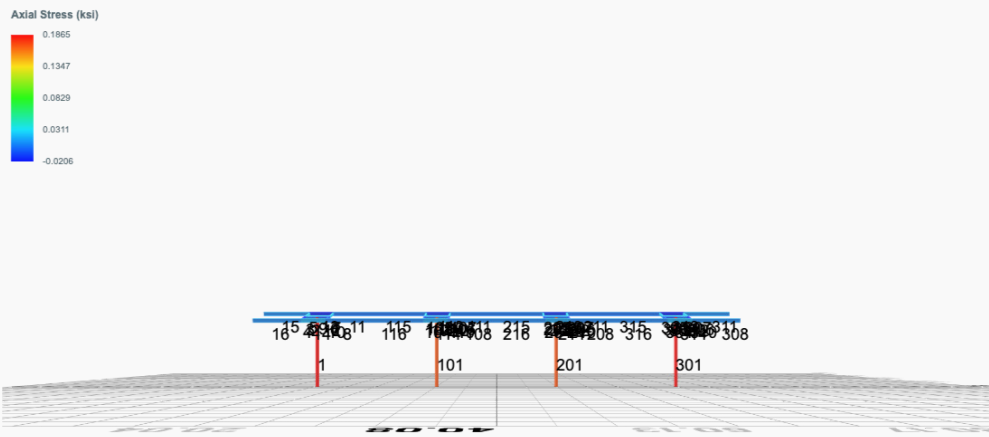




FEM Results (Envelope Worst Case for each member)







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 | -0.0079 | 2.4607 | -0.0498 | -0.1808 | 0.0204 | 0.1092 |
| ULS: 2. D + L | -0.0079 | 2.4607 | -0.0498 | -0.1808 | 0.0204 | 0.1092 |
| ULS: 3. D + (S or Lr or R) | -0.0119 | 3.5014 | -0.0752 | -0.2732 | 0.0308 | 0.1503 |
| ULS: 3. D + (S or Lr or R) | -0.0079 | 2.4607 | -0.0498 | -0.1808 | 0.0204 | 0.1092 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0109 | 3.2412 | -0.0689 | -0.2501 | 0.0282 | 0.1401 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0079 | 2.4607 | -0.0498 | -0.1808 | 0.0204 | 0.1092 |
| ULS: 5b. D + 0.7E | -0.0079 | 2.4607 | -0.0498 | -0.1808 | 0.0204 | 0.1092 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0109 | 3.2412 | -0.0689 | -0.2501 | 0.0282 | 0.1401 |
| ULS: 8. 0.6D + 0.7E | -0.0047 | 1.4764 | -0.0299 | -0.1085 | 0.0122 | 0.0655 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.9725 | 7.9888 | -0.1901 | -0.6922 | 0.0939 | 16.5735 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.9725 | 7.9888 | -0.1901 | -0.6922 | 0.0939 | 16.5735 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.6739 | -1.4174 | 0.0456 | 0.1632 | -0.0277 | -4.3222 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.5760 | -0.9122 | 0.0378 | 0.1350 | -0.0275 | -18.2525 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.7343 | 7.3873 | -0.1741 | -0.6336 | 0.0833 | 12.4883 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.7343 | 7.3873 | -0.1741 | -0.6336 | 0.0833 | 12.4883 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.5004 | 0.3327 | 0.0027 | 0.0079 | -0.0079 | -3.1835 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4270 | 0.7116 | -0.0032 | -0.0133 | -0.0078 | -13.6312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.7313 | 6.6068 | -0.1550 | -0.5643 | 0.0755 | 12.4575 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.7313 | 6.6068 | -0.1550 | -0.5643 | 0.0755 | 12.4575 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.5035 | -0.4479 | 0.0218 | 0.0772 | -0.0157 | -3.2144 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4300 | -0.0690 | 0.0159 | 0.0560 | -0.0156 | -13.6621 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.9693 | 7.0046 | -0.1702 | -0.6198 | 0.0857 | 16.5299 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.9693 | 7.0046 | -0.1702 | -0.6198 | 0.0857 | 16.5299 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.6771 | -2.4016 | 0.0655 | 0.2355 | -0.0359 | -4.3659 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.5792 | -1.8964 | 0.0577 | 0.2073 | -0.0357 | -18.2962 |

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 | 12.6868 |
| Shear X | -1.6178 |
| Shear Z | -0.3072 |
| Moment X | -1.1229 |
| Moment Y (Twist) | 0.1529 |
| Moment Z | 31.5225 |

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 | 7.9888 |
| Shear X | -0.9725 |
| Shear Z | -0.1901 |
| Moment X | -0.6922 |
| Moment Y (Twist) | 0.0939 |
| Moment Z | 18.2962 |

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 | 0.0079 | 2.3554 | 0.0119 | 0.0434 | -0.0051 | -0.0501 |
| ULS: 2. D + L | 0.0079 | 2.3554 | 0.0119 | 0.0434 | -0.0051 | -0.0501 |
| ULS: 3. D + (S or Lr or R) | 0.0119 | 3.3423 | 0.0179 | 0.0656 | -0.0077 | -0.0903 |
| ULS: 3. D + (S or Lr or R) | 0.0079 | 2.3554 | 0.0119 | 0.0434 | -0.0051 | -0.0501 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0109 | 3.0956 | 0.0164 | 0.0601 | -0.0071 | -0.0802 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0079 | 2.3554 | 0.0119 | 0.0434 | -0.0051 | -0.0501 |
| ULS: 5b. D + 0.7E | 0.0079 | 2.3554 | 0.0119 | 0.0434 | -0.0051 | -0.0501 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0109 | 3.0956 | 0.0164 | 0.0601 | -0.0071 | -0.0802 |
| ULS: 8. 0.6D + 0.7E | 0.0047 | 1.4132 | 0.0071 | 0.0261 | -0.0031 | -0.0301 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.9252 | 7.5892 | 0.0444 | 0.1633 | -0.0169 | 15.8721 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.9252 | 7.5892 | 0.0444 | 0.1633 | -0.0169 | 15.8721 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.6569 | -1.3139 | -0.0120 | -0.0431 | 0.0056 | -4.2722 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.5823 | -0.8407 | -0.0071 | -0.0260 | -0.0002 | -17.9665 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.6889 | 7.0209 | 0.0408 | 0.1500 | -0.0159 | 11.8614 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.6889 | 7.0209 | 0.0408 | 0.1500 | -0.0159 | 11.8614 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.4977 | 0.3436 | -0.0015 | -0.0049 | 0.0010 | -3.2468 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4417 | 0.6985 | 0.0022 | 0.0080 | -0.0034 | -13.5176 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.6919 | 6.2807 | 0.0363 | 0.1333 | -0.0139 | 11.8916 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.6919 | 6.2807 | 0.0363 | 0.1333 | -0.0139 | 11.8916 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.4946 | -0.3966 | -0.0060 | -0.0215 | 0.0030 | -3.2167 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4387 | -0.0417 | -0.0024 | -0.0086 | -0.0014 | -13.4874 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.9283 | 6.6470 | 0.0397 | 0.1460 | -0.0148 | 15.8922 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.9283 | 6.6470 | 0.0397 | 0.1460 | -0.0148 | 15.8922 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.6537 | -2.2561 | -0.0167 | -0.0605 | 0.0077 | -4.2521 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.5791 | -1.7829 | -0.0119 | -0.0433 | 0.0019 | -17.9465 |

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 | 12.0429 |
| Shear X | -1.5551 |
| Shear Z | 0.0714 |
| Moment X | 0.2635 |
| Moment Y (Twist) | 0.0265 |
| Moment Z | 30.9685 |

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 | 7.5892 |
| Shear X | -0.9283 |
| Shear Z | 0.0444 |
| Moment X | 0.1633 |
| Moment Y (Twist) | 0.0169 |
| Moment Z | 17.9665 |

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 | 0.0079 | 2.3554 | -0.0119 | -0.0434 | 0.0051 | -0.0501 |
| ULS: 2. D + L | 0.0079 | 2.3554 | -0.0119 | -0.0434 | 0.0051 | -0.0501 |
| ULS: 3. D + (S or Lr or R) | 0.0119 | 3.3423 | -0.0179 | -0.0656 | 0.0077 | -0.0903 |
| ULS: 3. D + (S or Lr or R) | 0.0079 | 2.3554 | -0.0119 | -0.0434 | 0.0051 | -0.0501 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0109 | 3.0956 | -0.0164 | -0.0601 | 0.0071 | -0.0802 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0079 | 2.3554 | -0.0119 | -0.0434 | 0.0051 | -0.0501 |
| ULS: 5b. D + 0.7E | 0.0079 | 2.3554 | -0.0119 | -0.0434 | 0.0051 | -0.0501 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0109 | 3.0956 | -0.0164 | -0.0601 | 0.0071 | -0.0802 |
| ULS: 8. 0.6D + 0.7E | 0.0047 | 1.4132 | -0.0071 | -0.0261 | 0.0031 | -0.0301 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.9252 | 7.5892 | -0.0444 | -0.1633 | 0.0169 | 15.8721 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.9252 | 7.5892 | -0.0444 | -0.1633 | 0.0169 | 15.8721 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.6569 | -1.3139 | 0.0120 | 0.0431 | -0.0057 | -4.2722 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.5823 | -0.8407 | 0.0071 | 0.0260 | 0.0002 | -17.9665 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.6889 | 7.0209 | -0.0408 | -0.1500 | 0.0159 | 11.8614 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.6889 | 7.0209 | -0.0408 | -0.1500 | 0.0159 | 11.8614 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.4977 | 0.3436 | 0.0015 | 0.0049 | -0.0010 | -3.2468 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4417 | 0.6985 | -0.0022 | -0.0080 | 0.0034 | -13.5176 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.6919 | 6.2807 | -0.0363 | -0.1333 | 0.0139 | 11.8916 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.6919 | 6.2807 | -0.0363 | -0.1333 | 0.0139 | 11.8916 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.4946 | -0.3966 | 0.0060 | 0.0215 | -0.0030 | -3.2167 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4387 | -0.0417 | 0.0024 | 0.0086 | 0.0014 | -13.4874 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.9283 | 6.6470 | -0.0397 | -0.1459 | 0.0148 | 15.8922 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.9283 | 6.6470 | -0.0397 | -0.1459 | 0.0148 | 15.8922 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.6537 | -2.2561 | 0.0167 | 0.0605 | -0.0077 | -4.2521 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.5791 | -1.7829 | 0.0119 | 0.0433 | -0.0019 | -17.9465 |

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 | 12.0429 |
| Shear X | -1.5551 |
| Shear Z | -0.0714 |
| Moment X | -0.2635 |
| Moment Y (Twist) | 0.0264 |
| Moment Z | 30.9686 |

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 | 7.5892 |
| Shear X | -0.9283 |
| Shear Z | -0.0444 |
| Moment X | -0.1633 |
| Moment Y (Twist) | 0.0169 |
| Moment Z | 17.9665 |

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 | -0.0079 | 2.4607 | 0.0498 | 0.1808 | -0.0204 | 0.1092 |
| ULS: 2. D + L | -0.0079 | 2.4607 | 0.0498 | 0.1808 | -0.0204 | 0.1092 |
| ULS: 3. D + (S or Lr or R) | -0.0119 | 3.5014 | 0.0752 | 0.2732 | -0.0308 | 0.1503 |
| ULS: 3. D + (S or Lr or R) | -0.0079 | 2.4607 | 0.0498 | 0.1808 | -0.0204 | 0.1092 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0109 | 3.2412 | 0.0689 | 0.2501 | -0.0282 | 0.1401 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0079 | 2.4607 | 0.0498 | 0.1808 | -0.0204 | 0.1092 |
| ULS: 5b. D + 0.7E | -0.0079 | 2.4607 | 0.0498 | 0.1808 | -0.0204 | 0.1092 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0109 | 3.2412 | 0.0689 | 0.2501 | -0.0282 | 0.1401 |
| ULS: 8. 0.6D + 0.7E | -0.0047 | 1.4764 | 0.0299 | 0.1085 | -0.0122 | 0.0655 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.9725 | 7.9888 | 0.1901 | 0.6922 | -0.0939 | 16.5735 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.9725 | 7.9888 | 0.1901 | 0.6922 | -0.0939 | 16.5735 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.6739 | -1.4174 | -0.0456 | -0.1632 | 0.0277 | -4.3222 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.5760 | -0.9122 | -0.0378 | -0.1350 | 0.0275 | -18.2525 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.7343 | 7.3873 | 0.1741 | 0.6336 | -0.0833 | 12.4883 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.7343 | 7.3873 | 0.1741 | 0.6336 | -0.0833 | 12.4883 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.5004 | 0.3327 | -0.0027 | -0.0079 | 0.0079 | -3.1835 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4270 | 0.7116 | 0.0032 | 0.0133 | 0.0077 | -13.6312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.7313 | 6.6068 | 0.1550 | 0.5643 | -0.0755 | 12.4575 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.7313 | 6.6068 | 0.1550 | 0.5643 | -0.0755 | 12.4575 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.5035 | -0.4479 | -0.0218 | -0.0772 | 0.0157 | -3.2144 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.4300 | -0.0690 | -0.0159 | -0.0560 | 0.0156 | -13.6621 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|--|---------|---------|---------|---------|---------|----------|
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.9693 | 7.0046 | 0.1702 | 0.6199 | -0.0857 | 16.5299 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.9693 | 7.0046 | 0.1702 | 0.6199 | -0.0857 | 16.5299 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.6771 | -2.4016 | -0.0655 | -0.2355 | 0.0359 | -4.3659 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.5792 | -1.8964 | -0.0577 | -0.2073 | 0.0357 | -18.2962 |

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 | 12.6868 |
| Shear X | -1.6178 |
| Shear Z | 0.3072 |
| Moment X | 1.1229 |
| Moment Y (Twist) | 0.1530 |
| Moment Z | 31.5231 |

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 | 7.9888 |
| Shear X | -0.9725 |
| Shear Z | 0.1901 |
| Moment X | 0.6922 |
| Moment Y (Twist) | 0.0939 |
| Moment Z | 18.2962 |

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Carport-Day-5x13
 Unit System: imperial



Design Input Information

| Design Factors | | | |
|----------------|----------|----------|----------|
| Φ_t | Φ_c | Φ_b | Φ_v |
| 0.9 | 0.9 | 0.9 | 0.9 |

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

Section Dimensions

| ID | Name | d (in) | t_w (in) | | | | | |
|----|-----------------|--------|------------|--|--|--|--|--|
| 2 | 2in Pipe Sch 80 | 2.38 | 0.22 | | | | | |
| 5 | 4in Pipe Sch 80 | 4.50 | 0.34 | | | | | |
| 7 | 6in Pipe Sch 40 | 6.63 | 0.28 | | | | | |

| ID | Name | d (in) | b (in) | t_w (in) | t_b (in) | r (in) | | |
|----|-------------|--------|--------|------------|------------|--------|--|--|
| 16 | HSS5x3x3/16 | 5.00 | 3.00 | 0.17 | 0.17 | 0.17 | | |

| ID | Name | d (in) | t_w (in) | b_t (in) | b_b (in) | t_t (in) | t_b (in) | r (in) |
|----|-------|--------|------------|------------|------------|------------|------------|--------|
| 19 | W8x10 | 7.89 | 0.17 | 3.94 | 3.94 | 0.20 | 0.20 | 0.30 |

| Section Properties | | | | | | | | |
|--------------------|------|----------------------|----------------------|-----------------------------|-----------------------------|--------------------------|-----------------------------|-----------------------------|
| ID | Name | A (in ²) | J (in ⁴) | I_{y0} (in ⁴) | I_{z0} (in ⁴) | I_w (in ⁶) | S_{y0} (in ³) | S_{z0} (in ³) |

| | | | | | | | | |
|-----|----|------|------|-------|---|------|------|---|
| 314 | 19 | 4.88 | 4.00 | 0 | 1,1.19,1.08,1.08,1.08,1.53 | 0 | 0 | 1 |
| 315 | 19 | 6.63 | 6.63 | 10.20 | 1.30,1.30,1.30,1.30,1.30,1.29,1.29,1.37,1.28,1.29,1.29,1.36,1.28,1.29,1.29,1.28,1.26,1.29,1.29,1.78,1.28,1.29,1.29,1.35,1.28 | 30.0 | 20.0 | 1 |
| 316 | 19 | 6.63 | 6.63 | 10.20 | 1.32,1.32,1.32,1.32,1.32,1.32,1.35,1.35,1.30,1.26,1.35,1.35,1.30,1.25,1.34,1.34,1.82,1.28,1.34,1.34,1.29,1.27,1.35,1.35,1.31,1.23 | 30.0 | 20.0 | 1 |

Member Design Capacity

| Member ID | $\Phi_t P_n$ (kip) | $\Phi_c P_n$ (kip) | $\Phi_b M_{zn}$ (k-ft) | $\Phi_b M_{yn}$ (k-ft) | $\Phi_v V_{yn}$ (kip) | $\Phi_v V_{zn}$ (kip) |
|-----------|--------------------|--------------------|------------------------|------------------------|-----------------------|-----------------------|
| 1 | 251.16 | 75.68 | 42.30 | 42.30 | 75.35 | 75.35 |
| 2 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 3 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 4 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 5 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 6 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 7 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 8 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |
| 9 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 10 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 11 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |
| 12 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 13 | 133.20 | 85.85 | 24.80 | 6.12 | 40.24 | 43.62 |
| 14 | 133.20 | 85.85 | 24.76 | 6.12 | 40.24 | 43.62 |
| 15 | 133.20 | 20.65 | 32.87 | 6.12 | 40.24 | 43.62 |
| 16 | 133.20 | 20.65 | 32.87 | 6.12 | 40.24 | 43.62 |
| 101 | 251.16 | 75.68 | 42.30 | 42.30 | 75.35 | 75.35 |
| 102 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 103 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 104 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 105 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 106 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 107 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 108 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |
| 109 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 110 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 111 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |
| 112 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 113 | 133.20 | 85.85 | 23.93 | 6.12 | 40.24 | 43.62 |
| 114 | 133.20 | 85.85 | 23.64 | 6.12 | 40.24 | 43.62 |
| 115 | 133.20 | 69.16 | 17.72 | 6.12 | 40.24 | 43.62 |
| 116 | 133.20 | 69.16 | 17.96 | 6.12 | 40.24 | 43.62 |
| 201 | 251.16 | 75.68 | 42.30 | 42.30 | 75.35 | 75.35 |
| 202 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 203 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 204 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 205 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 206 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 207 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 208 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |
| 209 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 210 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 211 | 133.20 | 123.95 | 32.87 | 6.12 | 40.24 | 43.62 |

| | | | | | | |
|-----|--------|--------|-------|-------|-------|-------|
| 212 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 213 | 133.20 | 85.85 | 23.92 | 6.12 | 40.24 | 43.62 |
| 214 | 133.20 | 85.85 | 23.64 | 6.12 | 40.24 | 43.62 |
| 215 | 133.20 | 69.16 | 16.94 | 6.12 | 40.24 | 43.62 |
| 216 | 133.20 | 69.16 | 16.05 | 6.12 | 40.24 | 43.62 |
| 301 | 251.16 | 75.68 | 42.30 | 42.30 | 75.35 | 75.35 |
| 302 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 303 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 304 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 305 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 306 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 307 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 308 | 133.20 | 20.65 | 32.87 | 6.12 | 40.24 | 43.62 |
| 309 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 310 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 311 | 133.20 | 20.65 | 32.87 | 6.12 | 40.24 | 43.62 |
| 312 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 313 | 133.20 | 85.85 | 24.80 | 6.12 | 40.24 | 43.62 |
| 314 | 133.20 | 85.85 | 24.76 | 6.12 | 40.24 | 43.62 |
| 315 | 133.20 | 69.16 | 19.50 | 6.12 | 40.24 | 43.62 |
| 316 | 133.20 | 69.16 | 19.04 | 6.12 | 40.24 | 43.62 |

Design Ratio

| Member ID | P | M _z | M _y | V _y | V _z | (P,M _z ,M _y) | Worst LC | KL/r | δ | Status |
|-----------|-------|----------------|----------------|----------------|----------------|-------------------------------------|----------|--------------|--------------|--------|
| 1 | 0.168 | 0.745 | 0.057 | 0.021 | 0.004 | 0.786 | #13 | 0.645 | Not Required | Pass |
| 2 | 0.001 | 0.565 | 0.078 | 0.112 | 0.015 | 0.644 | #13 | 0.053 | Not Required | Pass |
| 3 | 0.002 | 0.898 | 0.017 | 0.091 | 0.002 | 0.915 | #13 | 0.045 | Not Required | Pass |
| 4 | 0.002 | 0.673 | 0.035 | 0.068 | 0.007 | 0.678 | #13 | 0.080 | Not Required | Pass |
| 5 | 0.002 | 0.557 | 0.041 | 0.090 | 0.011 | 0.570 | #13 | 0.074 | Not Required | Pass |
| 6 | 0.002 | 0.785 | 0.008 | 0.078 | 0.003 | 0.792 | #13 | 0.045 | Not Required | Pass |
| 7 | 0.002 | 0.487 | 0.027 | 0.078 | 0.007 | 0.488 | #13 | 0.074 | Not Required | Pass |
| 8 | 0.001 | 0.092 | 0.029 | 0.044 | 0.003 | 0.121 | #13 | 0.095 | Not Required | Pass |
| 9 | 0.003 | 0.111 | 0.028 | 0.003 | 0.001 | 0.141 | #13 | 0.204 | Not Required | Pass |
| 10 | 0.001 | 0.583 | 0.038 | 0.059 | 0.009 | 0.618 | #13 | 0.080 | Not Required | Pass |
| 11 | 0.001 | 0.117 | 0.027 | 0.058 | 0.003 | 0.142 | #13 | 0.095 | Not Required | Pass |
| 12 | 0.001 | 0.456 | 0.071 | 0.097 | 0.013 | 0.528 | #13 | 0.053 | Not Required | Pass |
| 13 | 0.002 | 0.421 | 0.086 | 0.073 | 0.003 | 0.484 | #13 | 0.286 | Not Required | Pass |
| 14 | 0.002 | 0.332 | 0.086 | 0.055 | 0.003 | 0.375 | #13 | 0.190 | Not Required | Pass |
| 15 | 0.000 | 0.191 | 0.052 | 0.052 | 0.002 | 0.228 | #13 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.141 | 0.052 | 0.039 | 0.002 | 0.179 | #13 | Not Required | Not Required | Pass |
| 101 | 0.159 | 0.732 | 0.013 | 0.021 | 0.001 | 0.737 | #16 | 0.645 | Not Required | Pass |
| 102 | 0.000 | 0.460 | 0.068 | 0.097 | 0.013 | 0.529 | #13 | 0.035 | Not Required | Pass |
| 103 | 0.002 | 0.790 | 0.010 | 0.079 | 0.002 | 0.795 | #13 | 0.045 | Not Required | Pass |
| 104 | 0.002 | 0.578 | 0.026 | 0.058 | 0.005 | 0.591 | #13 | 0.080 | Not Required | Pass |
| 105 | 0.002 | 0.490 | 0.026 | 0.079 | 0.006 | 0.492 | #13 | 0.074 | Not Required | Pass |
| 106 | 0.002 | 0.814 | 0.013 | 0.082 | 0.003 | 0.826 | #13 | 0.045 | Not Required | Pass |
| 107 | 0.002 | 0.504 | 0.026 | 0.081 | 0.006 | 0.509 | #13 | 0.074 | Not Required | Pass |
| 108 | 0.001 | 0.065 | 0.022 | 0.039 | 0.002 | 0.072 | #13 | 0.095 | Not Required | Pass |
| 109 | 0.002 | 0.078 | 0.018 | 0.001 | 0.000 | 0.096 | #13 | 0.204 | Not Required | Pass |
| 110 | 0.002 | 0.604 | 0.024 | 0.061 | 0.005 | 0.612 | #13 | 0.080 | Not Required | Pass |

| | | | | | | | | | | |
|-----|-------|-------|-------|-------|-------|-------|-----|--------------|--------------|------|
| 110 | 0.002 | 0.004 | 0.024 | 0.001 | 0.003 | 0.012 | #13 | 0.000 | Not Required | Pass |
| 111 | 0.001 | 0.078 | 0.022 | 0.053 | 0.002 | 0.083 | #13 | 0.095 | Not Required | Pass |
| 112 | 0.000 | 0.486 | 0.069 | 0.101 | 0.013 | 0.555 | #13 | 0.053 | Not Required | Pass |
| 113 | 0.002 | 0.247 | 0.060 | 0.068 | 0.003 | 0.264 | #13 | 0.286 | Not Required | Pass |
| 114 | 0.002 | 0.184 | 0.060 | 0.051 | 0.003 | 0.196 | #13 | 0.286 | Not Required | Pass |
| 115 | 0.001 | 0.213 | 0.032 | 0.048 | 0.003 | 0.238 | #13 | 0.473 | Not Required | Pass |
| 116 | 0.001 | 0.162 | 0.032 | 0.035 | 0.003 | 0.186 | #13 | 0.473 | Not Required | Pass |
| 201 | 0.159 | 0.732 | 0.013 | 0.021 | 0.001 | 0.737 | #16 | 0.645 | Not Required | Pass |
| 202 | 0.000 | 0.486 | 0.069 | 0.101 | 0.013 | 0.555 | #13 | 0.053 | Not Required | Pass |
| 203 | 0.002 | 0.814 | 0.013 | 0.082 | 0.003 | 0.826 | #13 | 0.045 | Not Required | Pass |
| 204 | 0.002 | 0.604 | 0.024 | 0.061 | 0.005 | 0.612 | #13 | 0.080 | Not Required | Pass |
| 205 | 0.002 | 0.504 | 0.026 | 0.081 | 0.006 | 0.509 | #13 | 0.074 | Not Required | Pass |
| 206 | 0.002 | 0.790 | 0.010 | 0.079 | 0.002 | 0.795 | #13 | 0.045 | Not Required | Pass |
| 207 | 0.002 | 0.490 | 0.026 | 0.079 | 0.006 | 0.492 | #13 | 0.074 | Not Required | Pass |
| 208 | 0.001 | 0.044 | 0.024 | 0.035 | 0.003 | 0.057 | #13 | 0.095 | Not Required | Pass |
| 209 | 0.002 | 0.078 | 0.018 | 0.001 | 0.000 | 0.096 | #13 | 0.204 | Not Required | Pass |
| 210 | 0.002 | 0.578 | 0.026 | 0.058 | 0.005 | 0.591 | #13 | 0.080 | Not Required | Pass |
| 211 | 0.001 | 0.052 | 0.024 | 0.048 | 0.003 | 0.065 | #13 | 0.095 | Not Required | Pass |
| 212 | 0.000 | 0.460 | 0.068 | 0.097 | 0.013 | 0.529 | #13 | 0.035 | Not Required | Pass |
| 213 | 0.002 | 0.247 | 0.060 | 0.068 | 0.003 | 0.264 | #13 | 0.286 | Not Required | Pass |
| 214 | 0.002 | 0.184 | 0.060 | 0.051 | 0.003 | 0.196 | #13 | 0.286 | Not Required | Pass |
| 215 | 0.001 | 0.307 | 0.032 | 0.053 | 0.002 | 0.330 | #13 | 0.473 | Not Required | Pass |
| 216 | 0.002 | 0.243 | 0.032 | 0.039 | 0.002 | 0.267 | #13 | 0.473 | Not Required | Pass |
| 301 | 0.168 | 0.745 | 0.057 | 0.021 | 0.004 | 0.786 | #13 | 0.645 | Not Required | Pass |
| 302 | 0.001 | 0.456 | 0.071 | 0.097 | 0.013 | 0.528 | #13 | 0.053 | Not Required | Pass |
| 303 | 0.002 | 0.785 | 0.008 | 0.078 | 0.003 | 0.792 | #13 | 0.045 | Not Required | Pass |
| 304 | 0.001 | 0.583 | 0.038 | 0.059 | 0.009 | 0.618 | #13 | 0.080 | Not Required | Pass |
| 305 | 0.002 | 0.487 | 0.027 | 0.078 | 0.007 | 0.488 | #13 | 0.074 | Not Required | Pass |
| 306 | 0.002 | 0.898 | 0.017 | 0.091 | 0.002 | 0.915 | #13 | 0.045 | Not Required | Pass |
| 307 | 0.002 | 0.557 | 0.041 | 0.090 | 0.011 | 0.570 | #13 | 0.074 | Not Required | Pass |
| 308 | 0.000 | 0.141 | 0.052 | 0.039 | 0.002 | 0.179 | #13 | Not Required | Not Required | Pass |
| 309 | 0.003 | 0.112 | 0.028 | 0.003 | 0.001 | 0.141 | #13 | 0.204 | Not Required | Pass |
| 310 | 0.002 | 0.673 | 0.035 | 0.068 | 0.007 | 0.678 | #13 | 0.080 | Not Required | Pass |
| 311 | 0.000 | 0.191 | 0.052 | 0.052 | 0.002 | 0.228 | #13 | Not Required | Not Required | Pass |
| 312 | 0.001 | 0.565 | 0.078 | 0.112 | 0.015 | 0.644 | #13 | 0.053 | Not Required | Pass |
| 313 | 0.002 | 0.421 | 0.086 | 0.073 | 0.003 | 0.484 | #13 | 0.190 | Not Required | Pass |
| 314 | 0.002 | 0.332 | 0.086 | 0.055 | 0.003 | 0.375 | #13 | 0.286 | Not Required | Pass |
| 315 | 0.001 | 0.193 | 0.033 | 0.058 | 0.003 | 0.218 | #13 | 0.473 | Not Required | Pass |
| 316 | 0.001 | 0.145 | 0.032 | 0.044 | 0.003 | 0.174 | #13 | 0.473 | Not Required | Pass |

Definitions

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

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

| REFERENCES | CALCULATIONS | RESULTS |
|------------|--------------|---------|
|------------|--------------|---------|

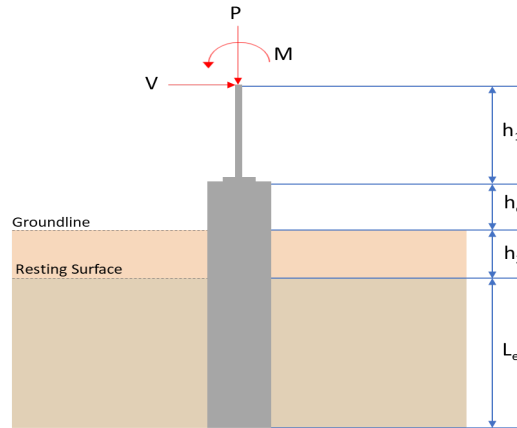
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 7.989 | 12.687 |
| V_x (kip) | -0.972 | -1.618 |
| V_z (kip) | -0.190 | -0.307 |
| M_x (kipft) | -0.692 | -1.123 |
| M_z (kipft) | 18.296 | 31.523 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.972 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.296 \text{ kipft}) + ((-0.972 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9134 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6523 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.19 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.030255 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.692 \text{ kipft}) + ((-0.19 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11019 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7751 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = MAX[(5.6523 \text{ ft}), (1.7751 \text{ ft})]$$

$$L_{e,req} = 5.652 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

Ratio - Embedded depth

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

$$Ratio = \frac{(5.652 \text{ ft})}{(6 \text{ ft})}$$

$$Ratio = 0.942$$

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

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

$$q = \frac{(7.989 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49931 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49931 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24966$$

Status: **PASS**
Ratio: **0.250**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.15478 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9134 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9134 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.9134 \text{ kipft/ft})) + (4 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0876 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9134 \text{ kipft/ft})) + (3 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.9134 \text{ kipft/ft})) + (2 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23801 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9134 \text{ kipft/ft})) + ((-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.81635 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0876 \text{ ft})}{2}$$

$$p_a = 0.30657 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.23801 \text{ kip/ft}^2)}{(0.30657 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77636$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.81635 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90705$$

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

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = -0.030255 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11019 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11019 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.030255 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.11019 \text{ kipft/ft})) + (4 \times (-0.030255 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2617 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11019 \text{ kipft/ft})) + (3 \times (-0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.11019 \text{ kipft/ft})) + (2 \times (-0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.006913 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.11019 \text{ kipft/ft})) + ((-0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.0064756 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2617 \text{ ft})}{2}$$

$$p_a = 0.31963 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(-0.006913 \text{ kip/ft}^2)}{(0.31963 \text{ kip/ft}^2)}$$

$$Ratio = -0.021628$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

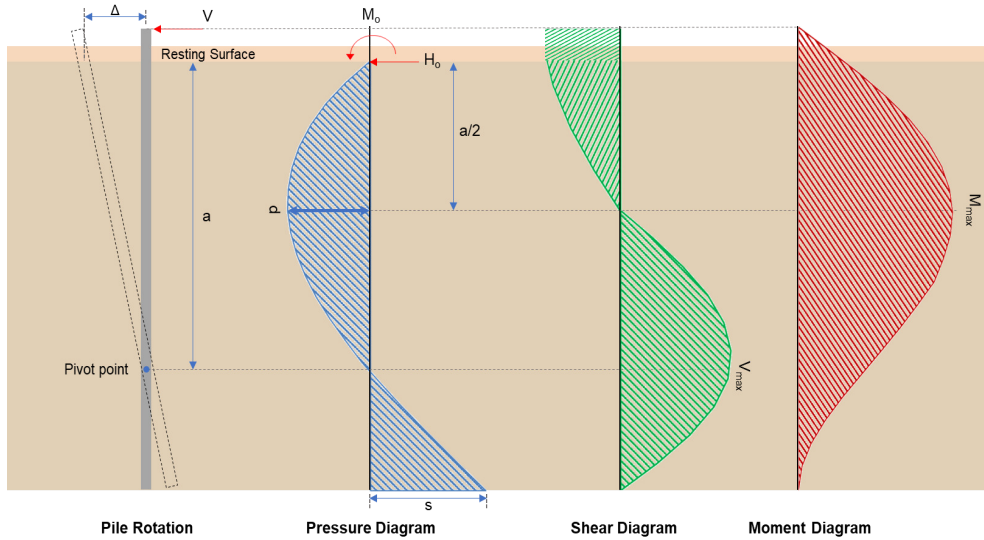
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0064756 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.0071951$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.618 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25764 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.523 \text{ kipft}) + ((-1.618 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0196 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0196 \text{ kipft/ft})}{(-0.25764 \text{ kip/ft})}$$

$$E = 19.483 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0196 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.0196 \text{ kipft/ft})) + (4 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.25764 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.0196 \text{ kip/ft})) + (4 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0852 \text{ ft}$$

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

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

$$V_{max} = ((-0.25764 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0852 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0852 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.6078 \text{ kip}$$

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

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

$$M_{max} = ((-0.25764 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(19.483 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.258 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.307 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.048885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.123 \text{ kipft}) + ((-0.307 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17882 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.17882 \text{ kipft/ft})}{(-0.048885 \text{ kip/ft})}$$

$$E = 3.658 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17882 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.048885 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.17882 \text{ kipft/ft})) + (4 \times (-0.048885 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2612 \text{ ft}$$

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

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

$$V_{max} = ((-0.048885 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2612 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2612 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.34085 \text{ kip}$$

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

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

$$M_{max} = ((-0.048885 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.658 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91762 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\left(\frac{12.687 \text{ kip}}{(0.65) \times (0.8)} \right) - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.175 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.175 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

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,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = \text{Min spacing of reinforcement,}$</p> $s_{rebar} = \text{Max}[1.5, (1.5 d_{bar})]$ $s_{rebar} = \text{Max}[1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>s_{ties} - Maximum spacing of ties,</p> $s_{ties} = \text{Min}[(16 d_{bar}), (48 d_{ties}), \text{Min}(D, b)]$ $s_{ties} = \text{Min}[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), \text{Min}((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 2675.2 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ $Ratio = \frac{(12.687 \text{ kip})}{(2675.2 \text{ kip})}$ $Ratio = 0.0047425$ | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> $d = 0.80 D$ $d = 0.80 \times (48 \text{ in})$ $d = 38.4 \text{ in}$ <p>λ_s - size effect modification factor</p> $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$ $\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$ $\lambda_s = 0.64282$ <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> $V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$ $V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.687 \text{ kip} \rightarrow 12687 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12687 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.18 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.18 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.18 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.18 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.2 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6078 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.6078 \text{ kip})}{(111.2 \text{ kip})}$$

$$Ratio = 0.059425$$

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

Considering z-direction:

$V_{max} = 0.34085 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.34085 \text{ kip})}{(111.2 \text{ kip})}$$

$$Ratio = 0.0030653$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 19.258 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.258 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.077155$$

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

Considering z-direction:

$M_{max} = 0.91762 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.91762 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0036764$$

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

| REFERENCES | CALCULATIONS | RESULTS |
|------------|--------------|---------|
|------------|--------------|---------|

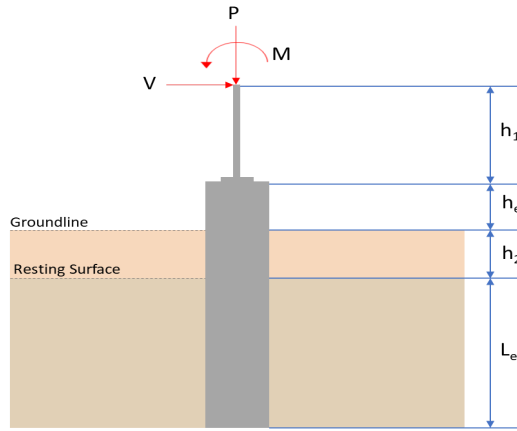
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular
 $b = 48$ in - Pile width
 $D = 48$ in - Pile depth
 $L = 6$ ft - Total pile length
 $h_1 = 0$ ft - Lateral load height from the top of the pile,
 $h_2 = 0$ ft - Depth to resisting surface
 $h_e = 0$ ft - Length of pile above the ground

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 7.589 | 12.043 |
| V_x (kip) | -0.928 | -1.555 |
| V_z (kip) | 0.044 | 0.071 |
| M_x (kipft) | 0.163 | 0.263 |
| M_z (kipft) | 17.967 | 30.968 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.928 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.967 \text{ kipft}) + ((-0.928 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.861 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6349 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.044 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0070064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.163 \text{ kipft}) + ((0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025955 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.3855 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6349 \text{ ft}), (1.3855 \text{ ft})]$$

$$L_{e,req} = 5.635 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.635 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.93917$$

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

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

$$q = \frac{(7.589 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23716$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.14777 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.861 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.861 \text{ kipft/ft})) + (4 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0856 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.861 \text{ kipft/ft})) + (3 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.861 \text{ kipft/ft})) + (2 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23606 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.861 \text{ kipft/ft})) + ((-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.80589 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0856 \text{ ft})}{2}$$

$$p_a = 0.30642 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23606 \text{ kip/ft}^2)}{(0.30642 \text{ kip/ft}^2)}$$

$$Ratio = 0.77038$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.80589 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.89544$$

Status: **PASS**
Ratio: **0.770**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = 0.0070064 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.025955 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.025955 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.0070064 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.025955 \text{ kipft/ft})) + (4 \times (0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2596 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.025955 \text{ kipft/ft})) + (3 \times (0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.025955 \text{ kipft/ft})) + (2 \times (0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.0068016 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.025955 \text{ kipft/ft})) + ((0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.015658 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2596 \text{ ft})}{2}$$

$$p_a = 0.31947 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.0068016 \text{ kip/ft}^2)}{(0.31947 \text{ kip/ft}^2)}$$

$$Ratio = 0.02129$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

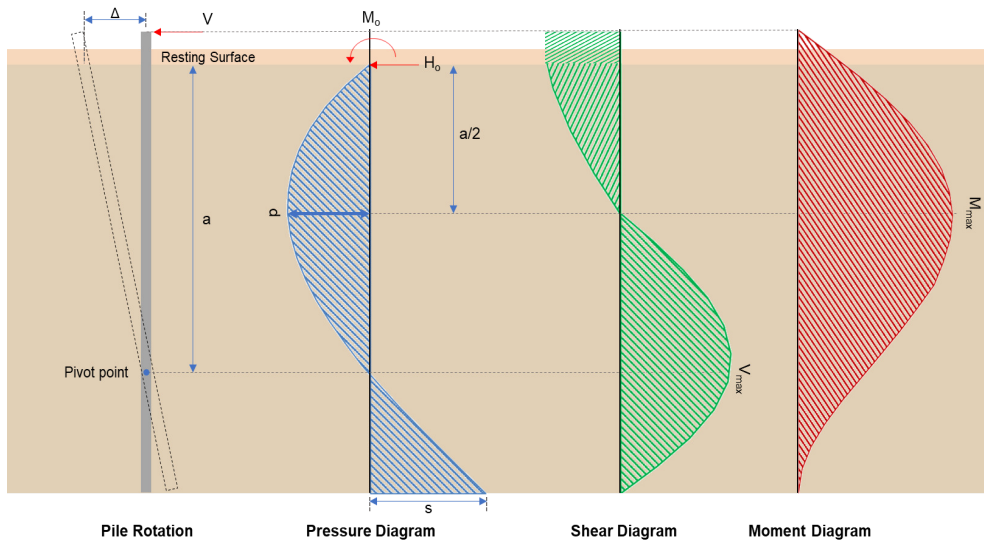
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.015658 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.017398$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.555 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.24761 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.968 \text{ kipft}) + ((-1.555 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9312 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.9312 \text{ kipft/ft})}{(-0.24761 \text{ kip/ft})}$$

$$E = 19.915 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9312 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.9312 \text{ kipft/ft})) + (4 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.24761 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.9312 \text{ kipft/ft})) + (4 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0836 \text{ ft}$$

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

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

$$V_{max} = ((-0.24761 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.915 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0836 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.915 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0836 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.4773 \text{ kip}$$

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

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

$$M_{max} = ((-0.24761 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(19.915 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.915 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.915 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.888 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.071 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.011306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.263 \text{ kipft}) + ((0.071 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.041879 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.041879 \text{ kipft/ft})}{(0.011306 \text{ kip/ft})}$$

$$E = 3.7042 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.041879 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.011306 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.041879 \text{ kipft/ft})) + (4 \times (0.011306 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2596 \text{ ft}$$

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

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

$$V_{max} = ((0.011306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2596 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2596 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.079441 \text{ kip}$$

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

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

$$M_{max} = ((0.011306 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.7042 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.21405 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.043 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.196 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.196 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(12.043 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0045017$</p> | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.043 \text{ kip} \rightarrow 12043 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12043 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.09 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(296.21 \text{ kip}), (120.09 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.09 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.09 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.14 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.4773 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.4773 \text{ kip})}{(111.14 \text{ kip})}$$

$$Ratio = 0.05828$$

Considering z-direction:

$V_{max} = 0.079441 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.079441 \text{ kip})}{(111.14 \text{ kip})}$$

$$Ratio = 0.00071478$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 18.888 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.888 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.075674$$

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

Considering z-direction:

$M_{max} = 0.21405 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.21405 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00085757$$

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

| REFERENCES | CALCULATIONS | RESULTS |
|------------|--------------|---------|
|------------|--------------|---------|

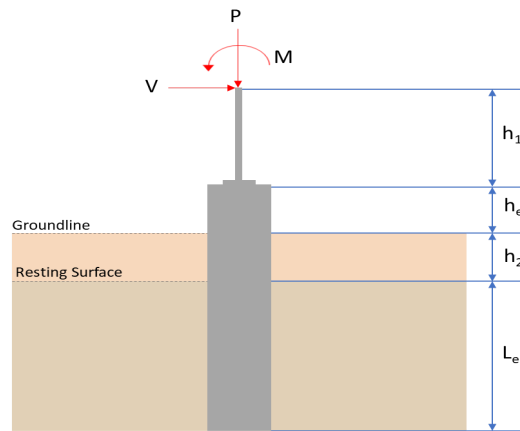
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 7.589 | 12.043 |
| V_x (kip) | -0.928 | -1.555 |
| V_z (kip) | -0.044 | -0.071 |
| M_x (kipft) | -0.163 | -0.263 |
| M_z (kipft) | 17.967 | 30.969 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.928 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.14777 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(17.967 \text{ kipft}) + ((-0.928 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.861 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6349 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.044 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0070064 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.163 \text{ kipft}) + ((-0.044 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.025955 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.1663 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6349 \text{ ft}), (1.1663 \text{ ft})]$$

$$L_{e,req} = 5.635 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.635 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.93917$$

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

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

$$q = \frac{(7.589 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.47431 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.47431 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.23716$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.14777 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.861 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.861 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.861 \text{ kipft/ft})) + (4 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0856 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.861 \text{ kipft/ft})) + (3 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.861 \text{ kipft/ft})) + (2 \times (-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23606 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.861 \text{ kipft/ft})) + ((-0.14777 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.80589 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0856 \text{ ft})}{2}$$

$$p_a = 0.30642 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23606 \text{ kip/ft}^2)}{(0.30642 \text{ kip/ft}^2)}$$

$$Ratio = 0.77038$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.80589 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.89544$$

Status: **PASS**
Ratio: **0.770**

Status: **PASS**
Ratio: **0.900**

Considering z-direction:

$H_o = -0.0070064 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.025955 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.025955 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.0070064 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.025955 \text{ kipft/ft})) + (4 \times (-0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2596 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.025955 \text{ kipft/ft})) + (3 \times (-0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 [(3 \times (0.025955 \text{ kipft/ft})) + (2 \times (-0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = -0.0016672 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.025955 \text{ kipft/ft})) + ((-0.0070064 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.0016454 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2596 \text{ ft})}{2}$$

$$p_a = 0.31947 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0016672 \text{ kip/ft}^2)}{(0.31947 \text{ kip/ft}^2)}$$

$$Ratio = -0.0052187$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

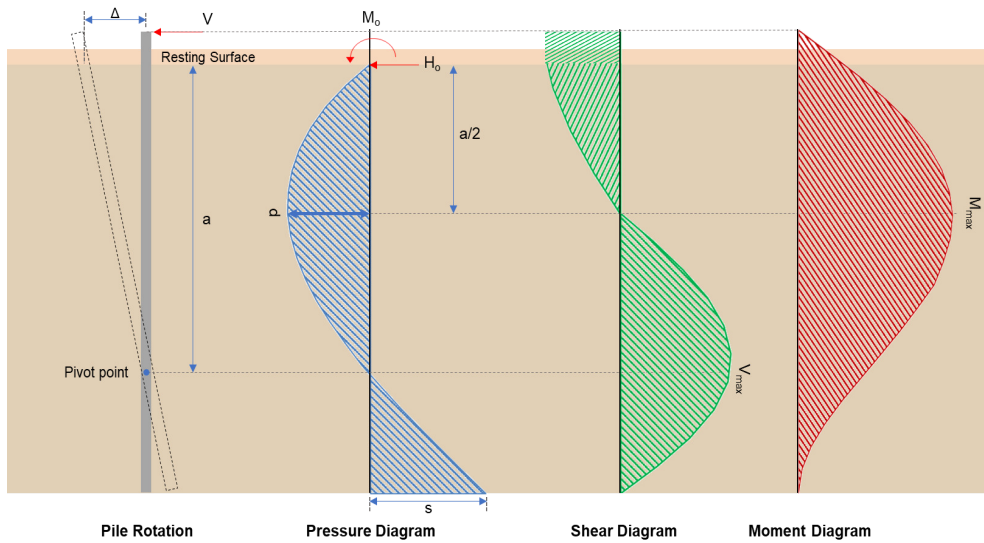
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.0016454 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.0018283$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.555 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.24761 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(30.969 \text{ kipft}) + ((-1.555 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 4.9314 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.9314 \text{ kipft/ft})}{(-0.24761 \text{ kip/ft})}$$

$$E = 19.916 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.9314 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (4.9314 \text{ kipft/ft})) + (4 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.24761 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (4.9314 \text{ kip/ft})) + (4 \times (-0.24761 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0836 \text{ ft}$$

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

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

$$V_{max} = ((-0.24761 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.916 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0836 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.916 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0836 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.4775 \text{ kip}$$

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

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

$$M_{max} = ((-0.24761 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(19.916 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.916 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.916 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0836 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 18.889 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.071 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.011306 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.263 \text{ kipft}) + ((-0.071 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.041879 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.041879 \text{ kipft/ft})}{(-0.011306 \text{ kip/ft})}$$

$$E = 3.7042 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.041879 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.011306 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.041879 \text{ kipft/ft})) + (4 \times (-0.011306 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2596 \text{ ft}$$

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

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

$$V_{max} = ((-0.011306 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2596 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] \right. \\ \left. + \left[4 \times \left(\frac{3 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2596 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.079441 \text{ kip}$$

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

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

$$M_{max} = ((-0.011306 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.7042 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right) \right. \\ \left. - \left[\left(\frac{4 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.7042 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2596 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.21405 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.043 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.196 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.196 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(12.043 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0045017$</p> | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.043 \text{ kip} \rightarrow 12043 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12043 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.09 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.09 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.09 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.09 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.14 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.4775 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(6.4775 \text{ kip})}{(111.14 \text{ kip})}$$

$$Ratio = 0.058282$$

Considering z-direction:

$V_{max} = 0.079441 \text{ kip}$ - Maximum shear force in the z-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.079441 \text{ kip})}{(111.14 \text{ kip})}$$

$$Ratio = 0.00071478$$

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

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 18.889 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(18.889 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.075676$$

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

Considering z-direction:

$M_{max} = 0.21405 \text{ kipft}$ - Maximum moment in the z-direction,

$Ratio$ - Capacity

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

$$\text{Ratio} = \frac{(0.21405 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$\text{Ratio} = 0.00085757$$

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

| REFERENCES | CALCULATIONS | RESULTS |
|------------|--------------|---------|
|------------|--------------|---------|

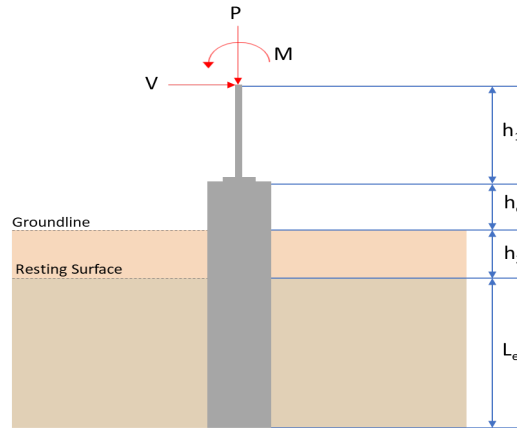
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 7.989 | 12.687 |
| V_x (kip) | -0.972 | -1.618 |
| V_z (kip) | 0.190 | 0.307 |
| M_x (kipft) | 0.692 | 1.123 |
| M_z (kipft) | 18.296 | 31.523 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.972 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.15478 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(18.296 \text{ kipft}) + ((-0.972 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 2.9134 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.6523 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.19 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.030255 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.692 \text{ kipft}) + ((0.19 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11019 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.3569 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.6523 \text{ ft}), (2.3569 \text{ ft})]$$

$$L_{e,req} = 5.652 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (6 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.652 \text{ ft})}{(6 \text{ ft})}$$

$$\text{Ratio} = 0.942$$

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

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

$$q = \frac{(7.989 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.49931 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.49931 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.24966$$

Status: **PASS**
Ratio: **0.250**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.15478 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.9134 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.9134 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (2.9134 \text{ kipft/ft})) + (4 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0876 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (2.9134 \text{ kipft/ft})) + (3 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (2.9134 \text{ kipft/ft})) + (2 \times (-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.23801 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (2.9134 \text{ kipft/ft})) + ((-0.15478 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.81635 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.0876 \text{ ft})}{2}$$

$$p_a = 0.30657 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.23801 \text{ kip/ft}^2)}{(0.30657 \text{ kip/ft}^2)}$$

$$Ratio = 0.77636$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.81635 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.90705$$

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

Status: **PASS**
Ratio: **0.910**

Considering z-direction:

$H_o = 0.030255 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.11019 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.11019 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.030255 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.11019 \text{ kipft/ft})) + (4 \times (0.030255 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2617 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.11019 \text{ kipft/ft})) + (3 \times (0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]^2}{(6 \text{ ft})^2 \times [(3 \times (0.11019 \text{ kipft/ft})) + (2 \times (0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]}$$

$$p = 0.029162 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.11019 \text{ kipft/ft})) + ((0.030255 \text{ kip/ft}) \times (6 \text{ ft}))]}{(6 \text{ ft})^2}$$

$$s = 0.066985 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2617 \text{ ft})}{2}$$

$$p_a = 0.31963 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.029162 \text{ kip/ft}^2)}{(0.31963 \text{ kip/ft}^2)}$$

$$Ratio = 0.091236$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6 \text{ ft})$$

$$p_s = 0.9 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

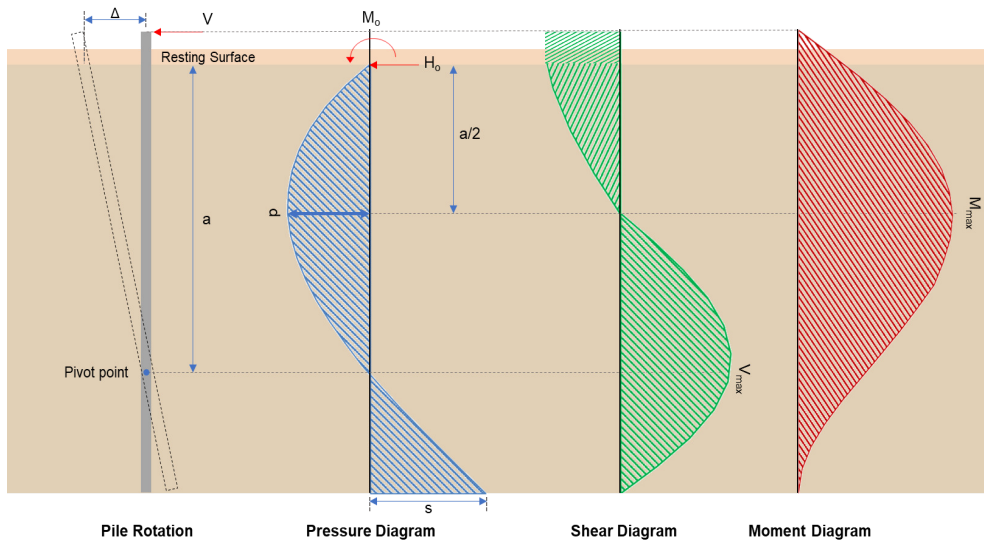
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.066985 \text{ kip/ft}^2)}{(0.9 \text{ kip/ft}^2)}$$

$$Ratio = 0.074428$$

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

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-1.618 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.25764 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(31.523 \text{ kipft}) + ((-1.618 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 5.0196 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.0196 \text{ kipft/ft})}{(-0.25764 \text{ kip/ft})}$$

$$E = 19.483 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.0196 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (5.0196 \text{ kipft/ft})) + (4 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = \frac{(-0.25764 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (5.0196 \text{ kip/ft})) + (4 \times (-0.25764 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.0852 \text{ ft}$$

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

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

$$V_{max} = ((-0.25764 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0852 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0852 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 6.6078 \text{ kip}$$

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

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

$$M_{max} = ((-0.25764 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(19.483 \text{ ft})}{(6 \text{ ft})} + \frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (19.483 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.0852 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 19.258 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.307 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.048885 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.123 \text{ kipft}) + ((0.307 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.17882 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.17882 \text{ kipft/ft})}{(0.048885 \text{ kip/ft})}$$

$$E = 3.658 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.17882 \text{ kipft/ft}) \times (6 \text{ ft})) + (3 \times (0.048885 \text{ kip/ft}) \times (6 \text{ ft})^2)}{(6 \times (0.17882 \text{ kipft/ft})) + (4 \times (0.048885 \text{ kip/ft}) \times (6 \text{ ft}))}$$

$$a = 4.2612 \text{ ft}$$

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

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

$$V_{max} = ((0.048885 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2612 \text{ ft})}{(6 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2612 \text{ ft})}{(6 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.34085 \text{ kip}$$

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

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

$$M_{max} = ((0.048885 \text{ kip/ft}) \times (48 \text{ in}) \times (6 \text{ ft})) \times \left[\left(\frac{(3.658 \text{ ft})}{(6 \text{ ft})} + \frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right) - \left[\left(\frac{4 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 3 \right) \times \left(\frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (3.658 \text{ ft})}{(6 \text{ ft})} + 2 \right) \times \left(\frac{(4.2612 \text{ ft})}{2 \times (6 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.91762 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.687 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.175 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.175 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(12.687 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0047425$</p> | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.687 \text{ kip} \rightarrow 12687 \text{ lbf}$, $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(12687 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.18 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.18 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.18 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((120.18 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 111.2 \text{ kip}$$

Considering x-direction:

$V_{max} = 6.6078 \text{ kip}$ - Maximum shear force in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(6.6078 \text{ kip})}{(111.2 \text{ kip})}$$

$$Ratio = 0.059425$$

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

Considering z-direction:

$V_{max} = 0.34085 \text{ kip}$ - Maximum shear force in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.34085 \text{ kip})}{(111.2 \text{ kip})}$$

$$Ratio = 0.0030653$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 19.258 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(19.258 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.077155$$

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

Considering z-direction:

$M_{max} = 0.91762 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(0.91762 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0036764$$

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