

Your Project Calculations



Project Name: D50Schools-Manor-JB-Area4

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=D50Schools-Manor-JB-Area4&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/9_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=NgKKaCE8GNyRTfp0p7FQEIkuCjsFEmH2poOChTIAImmt1UcBzNpSPlucmDejtjS

Array Specification

| | |
|------------------------------------|-----------------------------------|
| Product: | Beam |
| Unique ID: | 4P-19.75-6TOP-HD-45-L-5Hx10W-8EK8 |
| Duty Classification: | HD |
| Module Width: | 44.65 in |
| Module Length: | 89.69in |
| Number of Rows: | 5 |
| Number of Columns: | 10 |
| Total Number of Modules: | 50 |
| Desired Tilt Angle: | 10 |
| Front Edge Clearance: | 14 |
| Total Array Height at Tilt: | 17.25 ft |
| Total Frame Length: | 74.25 ft |
| Frame Weight: | 3531 lbs |
| Array Dimensions N/S: | 18.81 ft |
| Array Dimensions E/W: | 75.58 ft |
| Rail Length: | 225.75 in |
| Rail Spacing: | 3.78 ft |
| Rail Check: | PASS (69% utilized) |

Support Specifications

| | |
|---------------------------------|-----------------|
| Pole Size: | 6in Pipe Sch 40 |
| Pole Length above Grade: | 15.63 ft |
| Number of Poles: | 4 |
| Pole Spacing: | 19.75 ft |

Foundation Specifications

| | |
|--|--|
| Foundation Type: | Square |
| Foundation Dimensions: | 48 x 48 in |
| Foundation Depth (below grade): | Pile 1: 5.00 ft Pile 2: 5.00 ft Pile 3: 5.00 ft Pile 4: 5.00 ft |
| Foundation Volume: | 11.852 y ³ |
| Foundation Result: | PASSED |
| Mount Twist: | 0.054774 kip |

Site Info

| | |
|-----------------------------------|---------------------------|
| Risk Category: | I |
| Exposure: | B |
| Soil Classification: | sand |
| Site Location: | Washington, IL 61571, USA |
| Wind Speed: | 100 mph |
| Snow Load: | 20 psf |
| Design Uplift Pressure: | Multiple pressures |
| Design Downforce Pressure: | Multiple pressures |
| Design Snow Pressure: | 0.012096 ksf |



Design Disclaimer

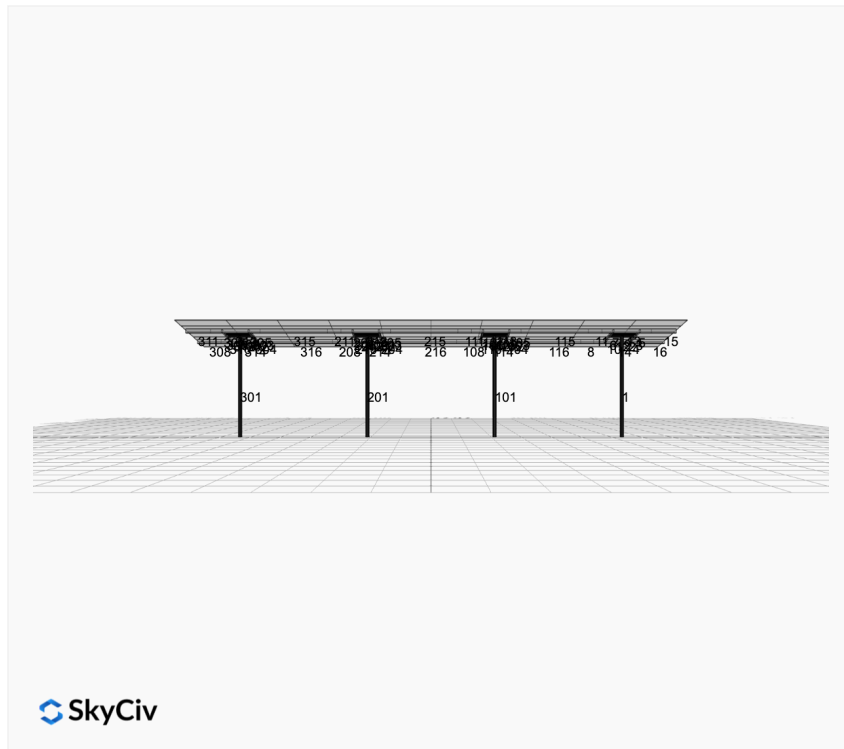
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

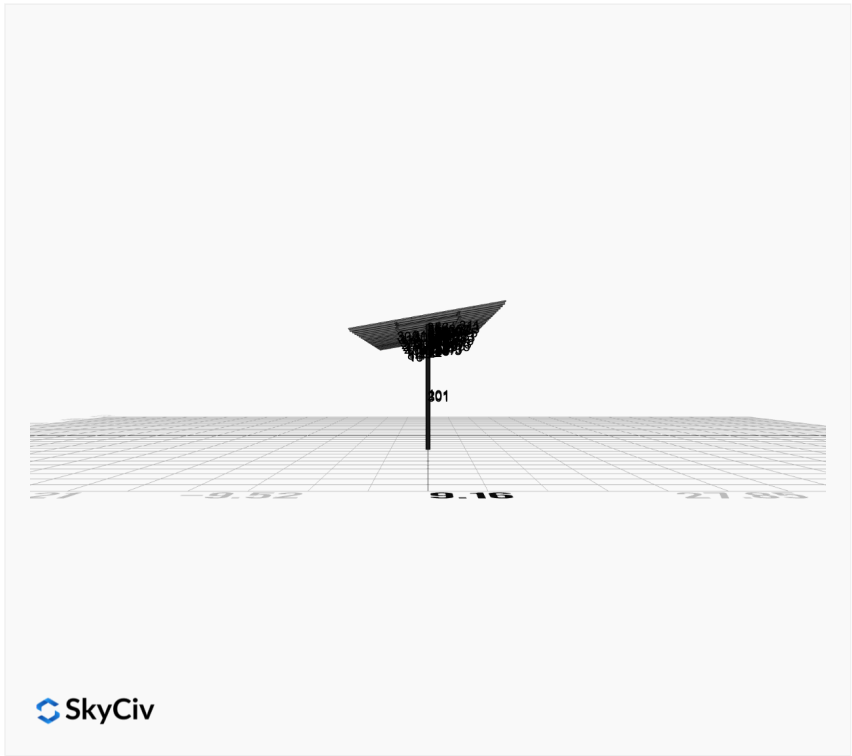
AutoDesigner Input

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{
  "wind_speed_override": null,
  "snow_load_override": null,
  "direct_snow_load": false,
  "add_angle_brace": false,
  "product_type": "Beam",
  "project_id": "D50Schools-Manor-JB-Area4",
  "site_address": "Washington, IL 61571, USA",
  "module_width": 44.65,
  "module_length": 89.69,
  "number_rows": 5,
  "number_columns": 10,
  "pole_mount_section": "4_40",
  "core_pipe_width": 65,
  "core_pipe_section": "2_40",
  "adjuster_section": "2_40",
  "core_beam_height": 65,
  "core_beam_section": "HSS3x2x1/8",
  "main_pipe_section": "2_12GA",
  "pole_spacing": 15,
  "tilt_angle": 10,
  "ground_clearance": 14,
  "risk_category": "I",
  "exposure_category": "B",
  "frame_duty_override": "auto",
  "pole_override": "auto",
  "soil_type": "sand",
  "customer_foundation_override": "48_Square",
  "foundation_type": "Square",
  "foundation_size": 48,
  "check_rails": true
}
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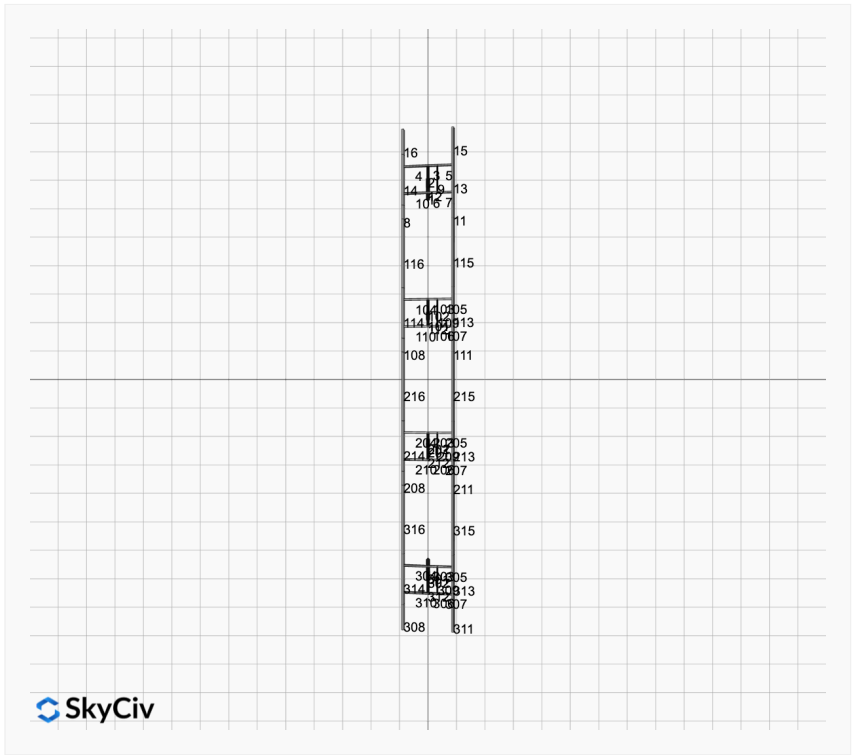
Design Notes:

- 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

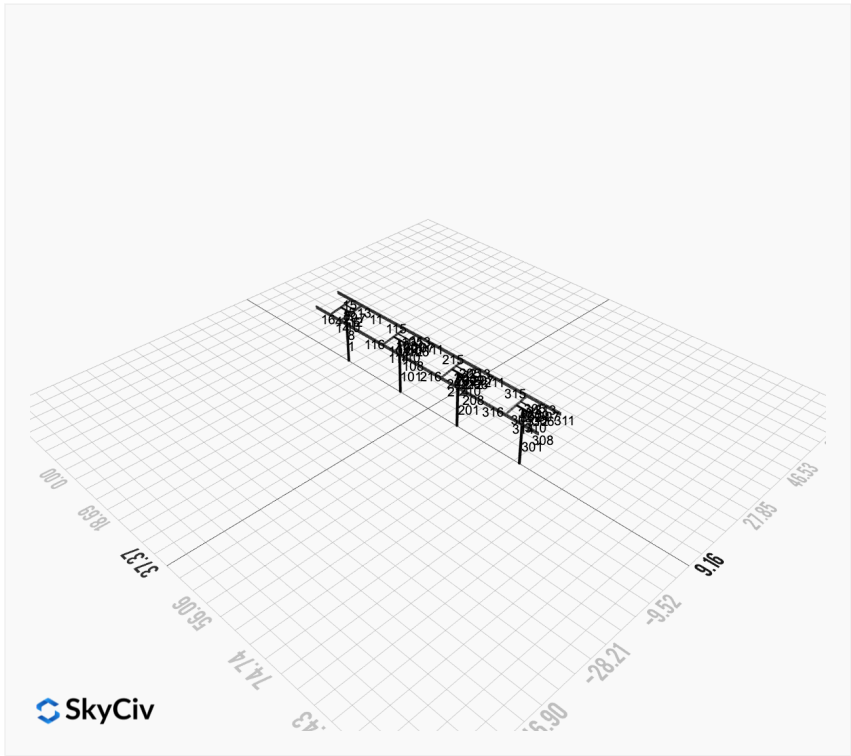




SkyCiv



SkyCiv



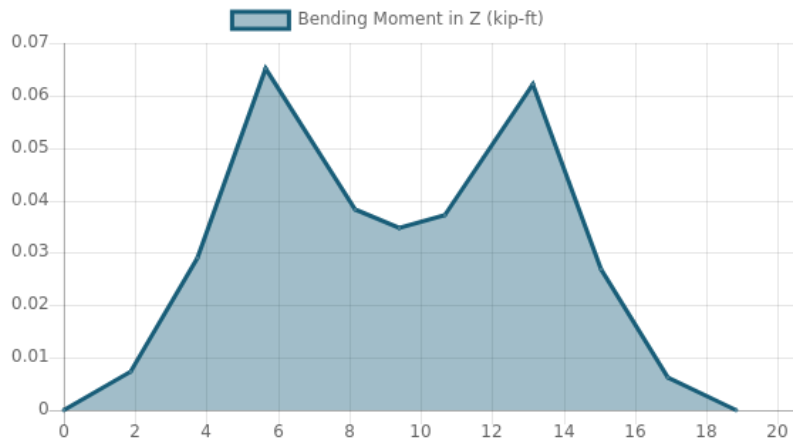
Rail Design Check

Rail Length: 18.8125 ft
Additional Restraints Required: None
Tributary Width: 3.77875 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Snow (X): 0.0450 kip/ft
Snow (Y): -0.0079 kip/ft
Wind uplift Case A: 0.0435 kip/ft
Wind uplift Case A: 0.0435 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0620 kip/ft



| Result Check | Max Limit | Max Value | Utility | Status |
|---------------------|-----------|-------------|---------|--------|
| Custom Stress Limit | 34.5 | 23.72751774 | 0.688 | PASS |
| Material Yield | 34.5 | 23.72751774 | 0.688 | PASS |
| Material Strength | 37 | 23.72751774 | 0.641 | PASS |

Member 1, ULS: 1. 1.4D



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 | 0.0024 | 2.3048 | 0.0178 | 0.0911 | -0.0034 | -0.0031 |
| ULS: 2. D + L | 0.0024 | 2.3048 | 0.0178 | 0.0911 | -0.0034 | -0.0031 |
| ULS: 3. D + (S or Lr or R) | 0.0075 | 6.1445 | 0.0560 | 0.2881 | -0.0109 | -0.0663 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.3048 | 0.0178 | 0.0911 | -0.0034 | -0.0031 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0062 | 5.1846 | 0.0465 | 0.2389 | -0.0090 | -0.0505 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.3048 | 0.0178 | 0.0911 | -0.0034 | -0.0031 |
| ULS: 5b. D + 0.7E | 0.0024 | 2.3048 | 0.0178 | 0.0911 | -0.0034 | -0.0031 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0062 | 5.1846 | 0.0465 | 0.2389 | -0.0090 | -0.0505 |
| ULS: 8. 0.6D + 0.7E | 0.0014 | 1.3829 | 0.0107 | 0.0547 | -0.0021 | -0.0018 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.4824 | 4.9122 | 0.0496 | 0.2540 | -0.0298 | 10.3313 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.4824 | 4.9122 | 0.0496 | 0.2540 | -0.0298 | 10.3313 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.3313 | 0.4783 | -0.0016 | -0.0070 | 0.0099 | -3.4788 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.3088 | 0.7114 | -0.0042 | -0.0201 | 0.0174 | -10.4967 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3574 | 7.1401 | 0.0704 | 0.3610 | -0.0288 | 7.7003 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3574 | 7.1401 | 0.0704 | 0.3610 | -0.0288 | 7.7003 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2529 | 3.8147 | 0.0320 | 0.1653 | 0.0010 | -2.6573 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2361 | 3.9895 | 0.0300 | 0.1554 | 0.0066 | -7.9207 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3612 | 4.2604 | 0.0417 | 0.2133 | -0.0232 | 7.7477 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3612 | 4.2604 | 0.0417 | 0.2133 | -0.0232 | 7.7477 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2491 | 0.9349 | 0.0033 | 0.0176 | 0.0066 | -2.6099 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2322 | 1.1098 | 0.0013 | 0.0077 | 0.0122 | -7.8733 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.4834 | 3.9903 | 0.0425 | 0.2175 | -0.0284 | 10.3325 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.4834 | 3.9903 | 0.0425 | 0.2175 | -0.0284 | 10.3325 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.3303 | -0.4436 | -0.0087 | -0.0434 | 0.0113 | -3.4776 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.3079 | -0.2105 | -0.0114 | -0.0566 | 0.0188 | -10.4954 |

Worst Case Reactions LRFD

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

| Result | Value (kip, kip-ft) |
|------------------|---------------------|
| Axial | 11.0827 |
| Shear X | -0.8090 |
| Shear Z | 0.1106 |
| Moment X | 0.5737 |
| Moment Y (Twist) | 0.0548 |
| Moment Z | 19.2093 |

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 | 7.1401 |
| Shear X | -0.4834 |
| Shear Z | 0.0704 |
| Moment X | 0.3610 |
| Moment Y (Twist) | 0.0298 |
| Moment Z | 10.4967 |

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 | -0.0024 | 2.6040 | -0.0021 | -0.0110 | 0.0007 | 0.0628 |
| ULS: 2. D + L | -0.0024 | 2.6040 | -0.0021 | -0.0110 | 0.0007 | 0.0628 |
| ULS: 3. D + (S or Lr or R) | -0.0075 | 7.0840 | -0.0067 | -0.0348 | 0.0021 | 0.1434 |
| ULS: 3. D + (S or Lr or R) | -0.0024 | 2.6040 | -0.0021 | -0.0110 | 0.0007 | 0.0628 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0062 | 5.9640 | -0.0056 | -0.0288 | 0.0017 | 0.1233 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0024 | 2.6040 | -0.0021 | -0.0110 | 0.0007 | 0.0628 |
| ULS: 5b. D + 0.7E | -0.0024 | 2.6040 | -0.0021 | -0.0110 | 0.0007 | 0.0628 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0062 | 5.9640 | -0.0056 | -0.0288 | 0.0017 | 0.1233 |
| ULS: 8. 0.6D + 0.7E | -0.0014 | 1.5624 | -0.0013 | -0.0066 | 0.0004 | 0.0377 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.5127 | 5.6400 | -0.0020 | -0.0110 | -0.0070 | 11.0837 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.5127 | 5.6400 | -0.0020 | -0.0110 | -0.0070 | 11.0837 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.3666 | 0.4727 | -0.0005 | -0.0026 | 0.0018 | -3.7570 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.2985 | 0.7526 | -0.0035 | -0.0176 | 0.0091 | -10.9085 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3889 | 8.2410 | -0.0055 | -0.0288 | -0.0040 | 8.3890 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3889 | 8.2410 | -0.0055 | -0.0288 | -0.0040 | 8.3890 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2705 | 4.3655 | -0.0044 | -0.0225 | 0.0026 | -2.7416 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2195 | 4.5755 | -0.0066 | -0.0338 | 0.0081 | -8.1052 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3851 | 4.8810 | -0.0021 | -0.0110 | -0.0051 | 8.3285 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3851 | 4.8810 | -0.0021 | -0.0110 | -0.0051 | 8.3285 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2743 | 1.0055 | -0.0009 | -0.0047 | 0.0015 | -2.8021 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2233 | 1.2155 | -0.0032 | -0.0160 | 0.0070 | -8.1657 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.5117 | 4.5984 | -0.0012 | -0.0066 | -0.0072 | 11.0586 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.5117 | 4.5984 | -0.0012 | -0.0066 | -0.0072 | 11.0586 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.3675 | -0.5689 | 0.0003 | 0.0018 | 0.0015 | -3.7821 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.2995 | -0.2889 | -0.0027 | -0.0132 | 0.0089 | -10.9336 |

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 | 12.8221 |
| Shear X | -0.8504 |
| Shear Z | -0.0122 |
| Moment X | -0.0628 |
| Moment Y (Twist) | 0.0178 |
| Moment Z | 20.3074 |

Worst Case Reactions ASD

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

| Result | Value (kip, kip-ft) |
|------------------|---------------------|
| Axial | 8.2410 |
| Shear X | -0.5127 |
| Shear Z | -0.0067 |
| Moment X | -0.0348 |
| Moment Y (Twist) | 0.0091 |
| Moment Z | 11.0837 |

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 | -0.0024 | 2.6040 | 0.0021 | 0.0110 | -0.0007 | 0.0628 |
| ULS: 2. D + L | -0.0024 | 2.6040 | 0.0021 | 0.0110 | -0.0007 | 0.0628 |
| ULS: 3. D + (S or Lr or R) | -0.0075 | 7.0840 | 0.0067 | 0.0348 | -0.0020 | 0.1434 |
| ULS: 3. D + (S or Lr or R) | -0.0024 | 2.6040 | 0.0021 | 0.0110 | -0.0007 | 0.0628 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0062 | 5.9640 | 0.0056 | 0.0288 | -0.0017 | 0.1233 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0024 | 2.6040 | 0.0021 | 0.0110 | -0.0007 | 0.0628 |
| ULS: 5b. D + 0.7E | -0.0024 | 2.6040 | 0.0021 | 0.0110 | -0.0007 | 0.0628 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0062 | 5.9640 | 0.0056 | 0.0288 | -0.0017 | 0.1233 |
| ULS: 8. 0.6D + 0.7E | -0.0014 | 1.5624 | 0.0013 | 0.0066 | -0.0004 | 0.0377 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.5127 | 5.6400 | 0.0020 | 0.0110 | 0.0070 | 11.0837 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.5127 | 5.6400 | 0.0020 | 0.0110 | 0.0070 | 11.0837 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.3666 | 0.4727 | 0.0005 | 0.0026 | -0.0018 | -3.7570 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.2985 | 0.7526 | 0.0035 | 0.0176 | -0.0091 | -10.9085 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3889 | 8.2410 | 0.0055 | 0.0288 | 0.0040 | 8.3890 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3889 | 8.2410 | 0.0055 | 0.0288 | 0.0040 | 8.3890 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2705 | 4.3655 | 0.0044 | 0.0225 | -0.0026 | -2.7416 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2195 | 4.5755 | 0.0066 | 0.0338 | -0.0081 | -8.1052 |

| 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 | -0.3851 | 4.8810 | 0.0021 | 0.0110 | 0.0051 | 8.3285 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3851 | 4.8810 | 0.0021 | 0.0110 | 0.0051 | 8.3285 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2743 | 1.0055 | 0.0009 | 0.0047 | -0.0015 | -2.8021 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2233 | 1.2155 | 0.0032 | 0.0160 | -0.0070 | -8.1657 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.5117 | 4.5984 | 0.0012 | 0.0066 | 0.0072 | 11.0586 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.5117 | 4.5984 | 0.0012 | 0.0066 | 0.0072 | 11.0586 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.3675 | -0.5689 | -0.0003 | -0.0018 | -0.0015 | -3.7821 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.2995 | -0.2889 | 0.0027 | 0.0132 | -0.0089 | -10.9336 |

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 | 12.8221 |
| Shear X | -0.8504 |
| Shear Z | 0.0122 |
| Moment X | 0.0629 |
| Moment Y (Twist) | 0.0178 |
| Moment Z | 20.3075 |

Worst Case Reactions ASD

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

| Result | Value (kip, kip-ft) |
|------------------|---------------------|
| Axial | 8.2410 |
| Shear X | -0.5127 |
| Shear Z | 0.0067 |
| Moment X | 0.0348 |
| Moment Y (Twist) | 0.0091 |
| Moment Z | 11.0837 |

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 | 0.0024 | 2.3048 | -0.0178 | -0.0911 | 0.0034 | -0.0031 |
| ULS: 2. D + L | 0.0024 | 2.3048 | -0.0178 | -0.0911 | 0.0034 | -0.0031 |
| ULS: 3. D + (S or Lr or R) | 0.0075 | 6.1445 | -0.0560 | -0.2881 | 0.0109 | -0.0663 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.3048 | -0.0178 | -0.0911 | 0.0034 | -0.0031 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0062 | 5.1846 | -0.0465 | -0.2389 | 0.0090 | -0.0505 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.3048 | -0.0178 | -0.0911 | 0.0034 | -0.0031 |
| ULS: 5b. D + 0.7E | 0.0024 | 2.3048 | -0.0178 | -0.0911 | 0.0034 | -0.0031 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0062 | 5.1846 | -0.0465 | -0.2389 | 0.0090 | -0.0505 |
| ULS: 8. 0.6D + 0.7E | 0.0014 | 1.3829 | -0.0107 | -0.0547 | 0.0021 | -0.0018 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.4824 | 4.9122 | -0.0496 | -0.2540 | 0.0298 | 10.3313 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.4824 | 4.9122 | -0.0496 | -0.2540 | 0.0298 | 10.3313 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.3313 | 0.4783 | 0.0016 | 0.0070 | -0.0099 | -3.4788 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.3088 | 0.7114 | 0.0042 | 0.0201 | -0.0174 | -10.4967 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3574 | 7.1401 | -0.0704 | -0.3610 | 0.0288 | 7.7003 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3574 | 7.1401 | -0.0704 | -0.3610 | 0.0288 | 7.7003 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2529 | 3.8147 | -0.0320 | -0.1653 | -0.0010 | -2.6573 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2361 | 3.9895 | -0.0300 | -0.1554 | -0.0066 | -7.9207 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.3612 | 4.2604 | -0.0417 | -0.2133 | 0.0232 | 7.7477 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.3612 | 4.2604 | -0.0417 | -0.2133 | 0.0232 | 7.7477 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2491 | 0.9349 | -0.0033 | -0.0176 | -0.0066 | -2.6099 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.2322 | 1.1098 | -0.0013 | -0.0077 | -0.0122 | -7.8733 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.4834 | 3.9903 | -0.0425 | -0.2175 | 0.0284 | 10.3325 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.4834 | 3.9903 | -0.0425 | -0.2175 | 0.0284 | 10.3325 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.3303 | -0.4436 | 0.0087 | 0.0434 | -0.0113 | -3.4776 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.3079 | -0.2105 | 0.0114 | 0.0566 | -0.0188 | -10.4954 |

Worst Case Reactions LRFD


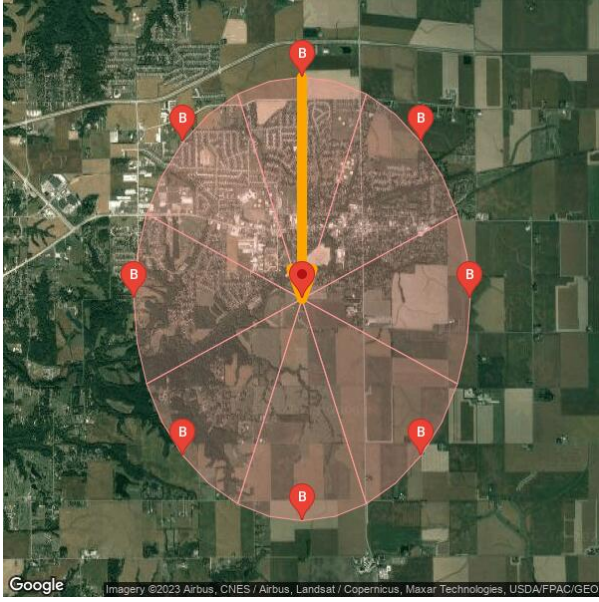
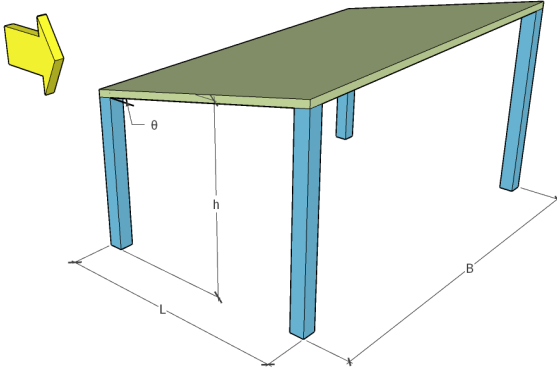
Worst Case Reactions ASD

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

| Result | Value (kip, kip-ft) |
|------------------|---------------------|
| Axial | 11.0827 |
| Shear X | -0.8090 |
| Shear Z | -0.1106 |
| Moment X | -0.5737 |
| Moment Y (Twist) | 0.0548 |
| Moment Z | 19.2096 |

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 | 7.1401 |
| Shear X | -0.4834 |
| Shear Z | -0.0704 |
| Moment X | -0.3610 |
| Moment Y (Twist) | 0.0298 |
| Moment Z | 10.4967 |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | |
|----------------------------|---|---------------------------------|-------|----------------------|----------|---------------------|----------|-----------------------|----------|--------------|----------------|----------------------------|--------|----------------|--|---------------|-------------|--|
| | <p style="text-align: center;">Wind Load Calculations based on ASCE 7-16</p> <p>Design Information :</p> <p>Project Name : D50Schools-Manor-JB-Area4 Client : Designer : MT SKYCIV AutoDesigner Company : MT Solar Units : Imperial Notes : Snow loads based on monoslope structure</p>  <p>Project Data</p> <p>The structure is located in Washington, IL 61571, USA categorized as Exposure B (assumed to be homogeneous for the selected wind direction). The wind load calculation for the structure - Main Wind Force Resisting System (MWFRS) - is based on the Directional Procedure (Chapter 27) of ASCE 7. Moreover, the structure is classified as Risk Category I. The location is elevated at 747.77 ft above mean sea level.</p>  <p style="text-align: center;">Figure 1. Site location.</p> <table border="1" data-bbox="609 1187 986 1370"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Building Length, L</td> <td>18.32 ft</td> </tr> <tr> <td>Building Width, B</td> <td>74.74 ft</td> </tr> <tr> <td>Mean Roof Height, h</td> <td>15.63 ft</td> </tr> <tr> <td>Roof Profile</td> <td>Open Monoslope</td> </tr> <tr> <td>Roof Pitch Angle, θ</td> <td>10.00°</td> </tr> <tr> <td>Structure Type</td> <td>Main Wind Force Resisting System (MWFRS)</td> </tr> <tr> <td>Wind Blockage</td> <td>Empty Under</td> </tr> </tbody> </table>  <p style="text-align: center;">Figure 2. Building parameters.</p> | Parameter | Value | Building Length, L | 18.32 ft | Building Width, B | 74.74 ft | Mean Roof Height, h | 15.63 ft | Roof Profile | Open Monoslope | Roof Pitch Angle, θ | 10.00° | Structure Type | Main Wind Force Resisting System (MWFRS) | Wind Blockage | Empty Under | |
| Parameter | Value | | | | | | | | | | | | | | | | | |
| Building Length, L | 18.32 ft | | | | | | | | | | | | | | | | | |
| Building Width, B | 74.74 ft | | | | | | | | | | | | | | | | | |
| Mean Roof Height, h | 15.63 ft | | | | | | | | | | | | | | | | | |
| Roof Profile | Open Monoslope | | | | | | | | | | | | | | | | | |
| Roof Pitch Angle, θ | 10.00° | | | | | | | | | | | | | | | | | |
| Structure Type | Main Wind Force Resisting System (MWFRS) | | | | | | | | | | | | | | | | | |
| Wind Blockage | Empty Under | | | | | | | | | | | | | | | | | |
| <p>Figure 26.5-1</p> | <p>Basic Wind Speed, V</p> <p>Wind speed for the address is 100 mph for Risk Category I and was calculated using Triangular Interpolation Network (TIN) method from points with known wind speed values based on Figure 26.5-1 of ASCE 7.</p> | <p>$V = 100$ mph</p> | | | | | | | | | | | | | | | | |
| <p>Table 26.6-1</p> | <p>Wind Directionality Factor, K_d</p> <p>$K_d = 0.85$ - Wind Directionality Factor For buildings</p> | <p>$K_d = 0.85$</p> | | | | | | | | | | | | | | | | |
| | <p>Topographic Factor, K_{zt}</p> | | | | | | | | | | | | | | | | | |

| Section 26.8.1 | <p>$K_{zt} = 1$ - Topographic Factor</p> <p>For the selected wind source direction, either the terrain is relatively a flat surface or the structure is outside the local topographic zones.</p> <p>For calculating the topographic factor, the detected topography for the selected wind source direction is Flat.</p> | $K_{zt} = 1$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|-------------------------|---|-------------------|-------------------|-------------------|-----------|-------------|-----------|-------------|----------|--------|--------|---------|--------|---------|--------|----------|-------|-------|---------|-------|---------|--------|-----------------------------|--------|-------|--------------------------------|--------|---------|--------|----|---|-------|-------|-------|---------|--------|---|---|-------|-------|-------|---------|--------|----|---|-------|-------|-------|---------|--------|---|---|-------|-------|-------|---------|--------|----|---|-------|-------|-------|---------|--------|--|
| Section 26.9 | <p>Ground Elevation Factor, K_e</p> <p>K_e - Ground Elevation Factor</p> $K_e = e^{-0.000362 E}$ $K_e = 0.97329$ <p>Where E = Site Elevation = 747.77 ft</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.9 | <p>$K_e = 0.97329$ - Ground Elevation Factor</p> | $K_e = 0.97329$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10 | <p>Velocity Pressure Exposure Coefficient, K_z</p> <p>K_z - Velocity Pressure Exposure Coefficient</p> <p>For $z < 15 ft$</p> $K_z = 2.01 \times (15/z_g)^{2/\alpha}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10 | <p>K_z - Velocity Pressure Exposure Coefficient</p> <p>For $15 ft \leq z \leq z_g$</p> $K_z = 2.01 \times (z/z_g)^{2/\alpha}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10 | <p>K_z - Velocity Pressure Exposure Coefficient</p> <p>For $15 ft \leq z \leq z_g$</p> $K_z = 2.01 (h/z_g)^{2/\alpha}$ $K_z = 0.58155$ <p>Where h = Mean Roof Height = 15.633 ft $z_g = 1200$ $\alpha = 7$</p> <table border="1"> <thead> <tr> <th>Level</th> <th>Elevation (ft)</th> <th>K_z</th> </tr> </thead> <tbody> <tr> <td>h</td> <td>15.633</td> <td>0.582</td> </tr> </tbody> </table> | Level | Elevation (ft) | K_z | h | 15.633 | 0.582 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Level | Elevation (ft) | K_z | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| h | 15.633 | 0.582 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10.2 | <p>Velocity Pressure, q_h</p> <p>For the selected wind source direction.</p> <p>q_h - Velocity Pressure at h</p> $q_h = 0.00256 K_z K_{zt} K_d K_e V^2$ $q_h = 12.317 \text{ psf}$ <p>Where K_z = Velocity Pressure Exposure Coefficient = 0.58155 K_{zt} = Topographic Factor = 1 K_d = Wind Directionality Factor = 0.85 V = Basic Wind Speed = 100 mi/h K_e = Ground Elevation Factor = 0.97329</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.8 | <p>Velocity Pressure for All Directions</p> <p>K_{zt} - Topographic Factor</p> $K_{zt} = (1 + K_1 \times K_2 \times K_3)^2$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10 | <p>K_z - Velocity Pressure Exposure Coefficient</p> <p>For $15 ft \leq z \leq z_g$</p> $K_z = 2.01 \times (z/z_g)^{2/\alpha}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Section 26.10 | <p>K_z - Velocity Pressure Exposure Coefficient</p> <p>For $z < 15 ft$</p> $K_z = 2.01 \times (15/z_g)^{2/\alpha}$ <table border="1"> <thead> <tr> <th>Direction</th> <th>Exposure Category</th> <th>@ $h = 15.633 ft$</th> <th>K_{zt}</th> <th>K_e</th> <th>V (mph)</th> <th>q_h (psf)</th> </tr> </thead> <tbody> <tr><td>N</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>NE</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>E</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>SE</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>S</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>SW</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>W</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> <tr><td>NW</td><td>B</td><td>0.582</td><td>1.000</td><td>0.973</td><td>100.000</td><td>12.317</td></tr> </tbody> </table> | Direction | Exposure Category | @ $h = 15.633 ft$ | K_{zt} | K_e | V (mph) | q_h (psf) | N | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | NE | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | E | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | SE | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | S | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | SW | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | W | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | NW | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | |
| Direction | Exposure Category | @ $h = 15.633 ft$ | K_{zt} | K_e | V (mph) | q_h (psf) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| N | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| NE | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| E | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| SE | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| S | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| SW | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| W | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| NW | B | 0.582 | 1.000 | 0.973 | 100.000 | 12.317 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Figure 27.3-4 to 27.3-7 | <p>Net Pressure Coefficients, C_N</p> <p>The net pressure coefficients, C_N, are calculated using Figures 27.3-4 to 27.3-7 of ASCE 7-16 - Clear Wind Flow - as shown in Table below.</p> <table border="1"> <thead> <tr> <th rowspan="2">Direction</th> <th rowspan="2">Surface</th> <th>C_N</th> <th>C_N</th> </tr> <tr> <th>Case A</th> <th>Case B</th> </tr> </thead> <tbody> <tr> <td rowspan="2">0</td> <td>Windward</td> <td>-0.700</td> <td>-1.567</td> </tr> <tr> <td>Leeward</td> <td>-1.100</td> <td>-</td> </tr> <tr> <td rowspan="2">180</td> <td>Windward</td> <td>1.033</td> <td>1.667</td> </tr> <tr> <td>Leeward</td> <td>1.533</td> <td>0.400</td> </tr> <tr> <td rowspan="2">90</td> <td>$\leq h$ from windward edge</td> <td>-0.800</td> <td>0.800</td> </tr> <tr> <td>h to $2h$ from windward edge</td> <td>-0.600</td> <td>0.500</td> </tr> </tbody> </table> | Direction | Surface | C_N | C_N | Case A | Case B | 0 | Windward | -0.700 | -1.567 | Leeward | -1.100 | - | 180 | Windward | 1.033 | 1.667 | Leeward | 1.533 | 0.400 | 90 | $\leq h$ from windward edge | -0.800 | 0.800 | h to $2h$ from windward edge | -0.600 | 0.500 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Direction | Surface | | | C_N | C_N | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | Case A | Case B | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0 | Windward | -0.700 | -1.567 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Leeward | -1.100 | - | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 180 | Windward | 1.033 | 1.667 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Leeward | 1.533 | 0.400 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 90 | $\leq h$ from windward edge | -0.800 | 0.800 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | h to $2h$ from windward edge | -0.600 | 0.500 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

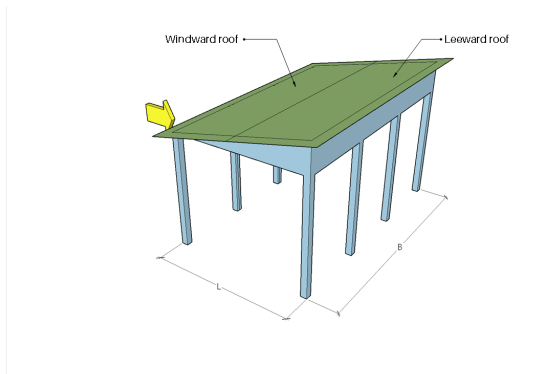
> 2h from windward edge | -0.300 | 0.300

| | | |
|--|---|------------|
| | Gust Effect Factor, G Section 26.11.1 $G = 0.85$ - Gust Effect Factor The structure is assumed to be rigid. | $G = 0.85$ |
|--|---|------------|

| | | |
|--|--|--|
| | Design Wind Pressures (MWFRS) Section 27.3.2 p - Design Wind Pressure For open buildings $p = q_h \times G \times C_N$ | |
|--|--|--|

For Wind Pressure - 0°

| Direction | Surface | q_h (psf) | G | C_N Case A | C_N Case B | p Case A (psf) | p Case B (psf) |
|-----------|----------|----------------|-------|-----------------|-----------------|------------------------|------------------------|
| 0 | Windward | 12.317 | 0.850 | -0.700 | -1.567 | -7.328 | -16.401 |
| | Leeward | 12.317 | 0.850 | -1.100 | - | -11.516 | 0.000 |
| 180 | Windward | 12.317 | 0.850 | 1.033 | 1.667 | 10.818 | 17.448 |
| | Leeward | 12.317 | 0.850 | 1.533 | 0.400 | 16.053 | 4.188 |



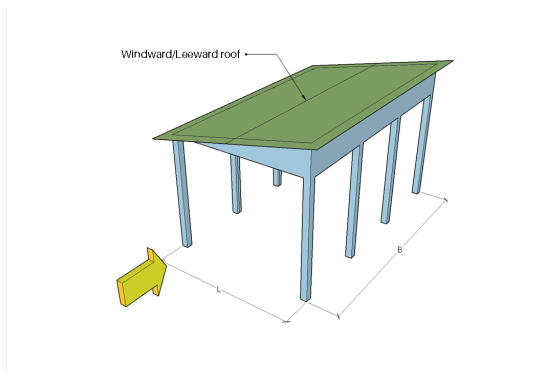
Wind along L - 0°

Service Wind Pressure - 0°/180°

| Direction | Surface | p Case A (psf) | p Case B (psf) |
|-----------|----------|------------------------|------------------------|
| 0 | Windward | -4.397 | -9.841 |
| | Leeward | -6.910 | 0.000 |
| 180 | Windward | 6.491 | 10.469 |
| | Leeward | 9.632 | 2.513 |

For Wind Pressure - 90°

| Direction | Surface | q_h (psf) | G | C_N Case A | C_N Case B | p Case A (psf) | p Case B (psf) |
|-----------|----------------------------|----------------|-------|-----------------|-----------------|------------------------|------------------------|
| 90 | ≤ h from windward edge | 12.317 | 0.850 | -0.800 | 0.800 | -8.375 | 8.375 |
| | h to 2h from windward edge | 12.317 | 0.850 | -0.600 | 0.500 | -6.281 | 5.235 |
| | > 2h from windward edge | 12.317 | 0.850 | -0.300 | 0.300 | -3.141 | 3.141 |



Wind along B - 90°

Service Wind Pressure - 90°

| Direction | Surface | p Case A (psf) | p Case B (psf) |
|-----------|----------------------------|------------------------|------------------------|
| 90 | ≤ h from windward edge | -5.025 | 5.025 |
| | h to 2h from windward edge | -3.769 | 3.141 |

| | | |
|-------------------------|--------|-------|
| > 2h from windward edge | -1.884 | 1.884 |
|-------------------------|--------|-------|

In addition to the roof pressures for 90°, an additional horizontal wind load on open building should be calculated for wind pressures parallel to the ridge in accordance with Section 28.3.5. We will assume $K_S = 1.0$ and should be adjusted based on the actual solidity ratio ϕ and number of frames n - See Figure 28.3-2.

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)

$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] \times K_B \times K_S$$

K_B - Frame Width Factor

For $L < 100ft$, $K_B = 1.8 - 0.01L$. Otherwise, $K_B = 0.8$.

$$K_B = 1.8 - 0.01 * L \leq 0.8$$

$$K_B = 1.6168$$

Where $L =$ Building Length = 18.322 ft

K_S - Shielding Factor

$$K_S = 0.6 + 0.073 \times (n - 1) + (1.25 \times \phi^{1.8})$$

$K_S = 1$ - Shielding Factor

Assumed to be equal to 1.0 and should be adjusted based on the actual wall solidity ratio ϕ and number of frames n .

$(GC_{pf})_{windward} = 0.4$

Using Zone 5 from Figure 28.3-1

$(GC_{pf})_{leeward} = -0.29$

Using Zone 6 from Figure 28.3-1

p - Horizontal Wind Loads on Open or Partially Enclosed Buildings

For wind pressure parallel to the ridge (90°)



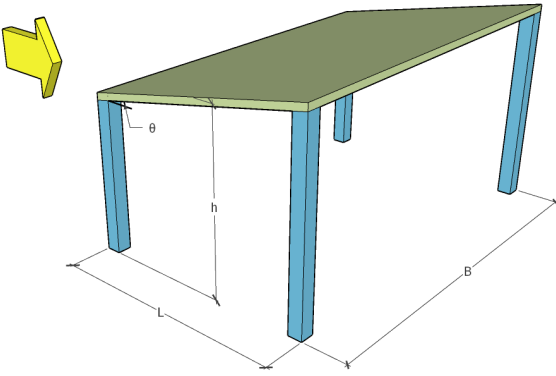
$$p = q_h \times [(GC_{pf})_{windward} - (GC_{pf})_{leeward}] K_B K_S$$

$$p = 13.74 \text{ psf}$$

$K_S = 1$

$(GC_{pf})_{windward} = 0.4$

$(GC_{pf})_{leeward} = -0.29$

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | |
|--|--|--|-------|---------------------------|-----------|--------------------------|-----------|----------------------------|-----------|--------------|----------------|----------------------------|---------|--|
| | <p style="text-align: center;">Snow Load Calculations based on ASCE 7-16</p> <p>Design Information :</p> <p>Project Name : D50Schools-Manor-JB-Area4 Client : Designer : MT SKYCIV AutoDesigner Company : MT Solar Units : Imperial Notes : Snow loads based on monoslope structure</p>  <p>Project Data</p> <p>The structure is located in Washington, IL 61571, USA categorized as Risk Category I. The snow load calculation for the structure is based on the Snow Loads (Chapter 7) of ASCE 7. The location is elevated at 748 ft above mean sea level.</p>  <p style="text-align: center;">Figure 1. Site location.</p> <table border="1" data-bbox="683 1055 911 1200"> <thead> <tr> <th>Parameter</th> <th>Value</th> </tr> </thead> <tbody> <tr> <td>Building Length, <i>L</i></td> <td>18.322 ft</td> </tr> <tr> <td>Building Width, <i>B</i></td> <td>74.742 ft</td> </tr> <tr> <td>Mean Roof Height, <i>h</i></td> <td>15.633 ft</td> </tr> <tr> <td>Roof Profile</td> <td>Open Monoslope</td> </tr> <tr> <td>Roof Pitch Angle, θ</td> <td>10.000°</td> </tr> </tbody> </table>  <p style="text-align: center;">Figure 2. Building parameters.</p> <p>Figure 7.6-2 Where roof slope is equal to $\frac{1}{S}$</p> $S = \frac{1}{\tan\theta}$ $S = 5.6713$ <p>Where θ = Angle of slope of roof = 10 °</p> | Parameter | Value | Building Length, <i>L</i> | 18.322 ft | Building Width, <i>B</i> | 74.742 ft | Mean Roof Height, <i>h</i> | 15.633 ft | Roof Profile | Open Monoslope | Roof Pitch Angle, θ | 10.000° | |
| Parameter | Value | | | | | | | | | | | | | |
| Building Length, <i>L</i> | 18.322 ft | | | | | | | | | | | | | |
| Building Width, <i>B</i> | 74.742 ft | | | | | | | | | | | | | |
| Mean Roof Height, <i>h</i> | 15.633 ft | | | | | | | | | | | | | |
| Roof Profile | Open Monoslope | | | | | | | | | | | | | |
| Roof Pitch Angle, θ | 10.000° | | | | | | | | | | | | | |
| <p>From Fig. 7.2-1 and Table 7.2-1 ATC/ASCE 7 Hazard Tools</p> <p>Google Maps</p> | <p>Ground Snow Load, p_g</p> <p>$p_g = 20$ psf - Ground Snow Load</p> <p>At elevation 747.77 ft above mean sea level.</p> <p>$E = 747.77$ ft - Ground Elevation above mean sea level</p> | <p>$p_g = 20$ psf</p> <p>$E = 747.77$ ft</p> | | | | | | | | | | | | |

| | | |
|---|--|---|
| Table 7.3-1 of ASCE 7-16 | <p>Exposure Factor, C_e</p> <p>$C_e = 0.9$ - Exposure Factor For Terrain Category Surface Roughness B with exposure condition specified as Fully Exposed.</p> | $C_e = 0.9$ |
| Table 7.3-2 of ASCE 7-16 | <p>Thermal Factor, C_t</p> <p>$C_t = 1.2$ - Thermal Factor For Thermal Condition equal to Unheated and open air structures.</p> | $C_t = 1.2$ |
| Table 1.5-1 of ASCE 7-16 | <p>Importance Factor, I_s</p> <p>$I_s = 0.8$ - Importance Factor For Risk Category I.</p> | $I_s = 0.8$ |
| Section 7.3 Equation 7.3-1 | <p>Flat Roof Snow Load, p_f</p> <p>p_f - Flat Roof Snow Load</p> $p_f = 0.7 C_e C_t I_s p_g$ $p_f = 12.096 \text{ psf}$ <p>Where C_e = Exposure Factor = 0.9 C_t = Thermal Factor = 1.2 I_s = Importance Factor = 0.8 p_g = Ground Snow Load = 20 psf</p> | |
| Section 7.10 | <p>Rain-on-snow Surcharge Load, p_r</p> <p>p_r - Rain-on-snow Surcharge Load For $0 < p_g \leq 20$ and $\theta < \frac{W}{50}$, $p_r = 5$ psf. Otherwise, $p_r = 0$ psf. This applies only to sloped roof (balanced) snow load case.</p> $p_r = 5 : 0 < p_g \leq 20, \theta < \frac{W}{50}$ $0 : \text{all cases}$ $p_r = 0 \text{ psf}$ <p>Where p_g = Ground Snow Load = 20 psf θ = Angle of slope of roof = 10° W = Horizontal distance from eave to ridge - equal to L for monoslope/monopitch roof. = 18.322 ft</p> | |
| Equation 7.7-1 | <p>Snow Density, γ</p> <p>γ - Snow Density</p> $\gamma = 0.13 p_g + 14 \leq 30$ $\gamma = 16.6 \text{ lbf/ft}^3$ <p>Where p_g = Ground Snow Load = 20 psf</p> | |
| Table 7.3-2 of ASCE 7-16 Figure 7-2c | <p>Roof Slope Factor (Balanced), C_s</p> <p>$C_t = 1.2$ - Thermal Factor For Thermal Condition equal to Unheated and open air structures.</p> <p>$\theta = 10^\circ$ - Angle of slope of roof</p> <p>$C_s = 1$ - Roof Slope Factor For roof pitch angle equal to 10.000° and Thermal Condition equal to Unheated and open air structures.</p> | $C_t = 1.2$ $\theta = 10^\circ$ $C_s = 1$ |
| Section 7.4 | <p>Sloped Roof Snow Load, p_s</p> <p>p_s - Sloped Roof Snow Load Assumed to act on the horizontal projection of the surface</p> $p_s = C_s p_f$ $p_s = 12.096 \text{ psf}$ <p>Where p_f = Flat Roof Snow Load = 12.096 psf C_s = Roof Slope Factor = 1</p> | |
| Section 7.3.4 | <p>Minimum Roof Snow Load, p_m</p> <p>p_m - Minimum Roof Snow Load For monoslope, hip, and gable roofs with slopes less than 15°</p> $p_m = I_s p_g \leq 20 p_g$ $p_m = 16 \text{ psf}$ <p>Where I_s = Importance Factor = 0.8 p_g = Ground Snow Load = 20 psf</p> | |

Project Details

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

User Name: sales@mtsolar.us
 Project Name: D50Schools-Manor-JB-Area4
 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 _{yp} (in ⁴) | I _{zp} (in ⁴) | I _w (in ⁶) | S _{yp} (in ³) | S _{zp} (in ³) |
| 2 | 2in Pipe Sch 80 | 1.48 | 1.74 | 0.87 | 0.87 | 0.00 | 1.02 | 1.02 |
| 5 | 4in Pipe Sch 80 | 4.41 | 19.22 | 9.61 | 9.61 | 0.00 | 5.85 | 5.85 |

| | | | | | | |
|-----|--------|--------|-------|-------|-------|-------|
| 103 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 104 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 105 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 106 | 116.10 | 115.41 | 15.79 | 11.10 | 42.08 | 23.28 |
| 107 | 116.10 | 114.23 | 15.79 | 11.10 | 42.08 | 23.28 |
| 108 | 133.20 | 126.01 | 32.87 | 6.12 | 40.24 | 43.62 |
| 109 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 110 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 111 | 133.20 | 126.01 | 32.87 | 6.12 | 40.24 | 43.62 |
| 112 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 113 | 133.20 | 104.94 | 23.83 | 6.12 | 40.24 | 43.62 |
| 114 | 133.20 | 104.94 | 23.37 | 6.12 | 40.24 | 43.62 |
| 115 | 133.20 | 69.16 | 17.02 | 6.12 | 40.24 | 43.62 |
| 116 | 133.20 | 69.16 | 17.18 | 6.12 | 40.24 | 43.62 |
| 201 | 251.16 | 40.96 | 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 | 126.01 | 32.87 | 6.12 | 40.24 | 43.62 |
| 209 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 210 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 211 | 133.20 | 126.01 | 32.87 | 6.12 | 40.24 | 43.62 |
| 212 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 213 | 133.20 | 104.94 | 23.83 | 6.12 | 40.24 | 43.62 |
| 214 | 133.20 | 104.94 | 23.37 | 6.12 | 40.24 | 43.62 |
| 215 | 133.20 | 69.16 | 17.02 | 6.12 | 40.24 | 43.62 |
| 216 | 133.20 | 69.16 | 17.49 | 6.12 | 40.24 | 43.62 |
| 301 | 251.16 | 40.96 | 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 | 52.83 | 32.87 | 6.12 | 40.24 | 43.62 |
| 309 | 66.48 | 58.89 | 3.82 | 3.82 | 19.94 | 19.94 |
| 310 | 116.10 | 111.33 | 15.79 | 11.10 | 42.08 | 23.28 |
| 311 | 133.20 | 52.83 | 32.87 | 6.12 | 40.24 | 43.62 |
| 312 | 198.33 | 196.72 | 21.95 | 21.95 | 59.50 | 59.50 |
| 313 | 133.20 | 104.94 | 24.52 | 6.12 | 40.24 | 43.62 |
| 314 | 133.20 | 104.94 | 24.52 | 6.12 | 40.24 | 43.62 |
| 315 | 133.20 | 69.16 | 17.02 | 6.12 | 40.24 | 43.62 |
| 316 | 133.20 | 69.16 | 17.02 | 6.12 | 40.24 | 43.62 |

Design Ratio

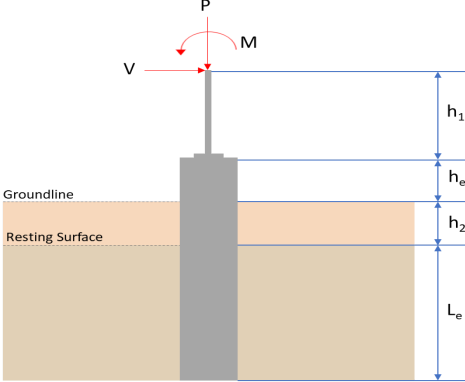
| Member ID | P | M _z | M _y | V _y | V _z | (P,M _z ,M _y) | Worst LC | KL/r | δ | Status |
|-----------|-------|----------------|----------------|----------------|----------------|-------------------------------------|----------|-------|--------------|--------|
| 1 | 0.271 | 0.454 | 0.027 | 0.011 | 0.001 | 0.628 | #13 | 0.877 | Not Required | Pass |
| 2 | 0.001 | 0.402 | 0.040 | 0.086 | 0.006 | 0.436 | #21 | 0.035 | Not Required | Pass |
| 3 | 0.003 | 0.623 | 0.013 | 0.062 | 0.002 | 0.638 | #21 | 0.045 | Not Required | Pass |

| | | | | | | | | | | |
|-----|-------|-------|-------|-------|-------|-------|-----|--------------|--------------|------|
| 4 | 0.003 | 0.572 | 0.045 | 0.058 | 0.010 | 0.608 | #21 | 0.080 | Not Required | Pass |
| 5 | 0.003 | 0.386 | 0.042 | 0.062 | 0.011 | 0.394 | #21 | 0.074 | Not Required | Pass |
| 6 | 0.004 | 0.681 | 0.029 | 0.069 | 0.006 | 0.710 | #21 | 0.045 | Not Required | Pass |
| 7 | 0.004 | 0.422 | 0.067 | 0.068 | 0.017 | 0.438 | #21 | 0.074 | Not Required | Pass |
| 8 | 0.001 | 0.082 | 0.061 | 0.042 | 0.006 | 0.115 | #21 | 0.095 | Not Required | Pass |
| 9 | 0.004 | 0.074 | 0.023 | 0.001 | 0.001 | 0.098 | #21 | 0.204 | Not Required | Pass |
| 10 | 0.004 | 0.617 | 0.061 | 0.062 | 0.013 | 0.647 | #21 | 0.080 | Not Required | Pass |
| 11 | 0.001 | 0.085 | 0.062 | 0.047 | 0.006 | 0.116 | #21 | 0.095 | Not Required | Pass |
| 12 | 0.001 | 0.455 | 0.042 | 0.094 | 0.007 | 0.490 | #21 | 0.053 | Not Required | Pass |
| 13 | 0.002 | 0.207 | 0.149 | 0.060 | 0.008 | 0.290 | #21 | 0.286 | Not Required | Pass |
| 14 | 0.002 | 0.192 | 0.147 | 0.055 | 0.008 | 0.275 | #21 | 0.190 | Not Required | Pass |
| 15 | 0.000 | 0.067 | 0.048 | 0.029 | 0.004 | 0.116 | #21 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.062 | 0.048 | 0.027 | 0.004 | 0.110 | #21 | Not Required | Not Required | Pass |
| 101 | 0.313 | 0.480 | 0.003 | 0.011 | 0.000 | 0.682 | #13 | 0.877 | Not Required | Pass |
| 102 | 0.001 | 0.501 | 0.046 | 0.105 | 0.006 | 0.541 | #21 | 0.053 | Not Required | Pass |
| 103 | 0.004 | 0.755 | 0.024 | 0.076 | 0.004 | 0.781 | #21 | 0.045 | Not Required | Pass |
| 104 | 0.004 | 0.701 | 0.066 | 0.070 | 0.014 | 0.746 | #21 | 0.080 | Not Required | Pass |
| 105 | 0.004 | 0.468 | 0.069 | 0.075 | 0.017 | 0.486 | #21 | 0.074 | Not Required | Pass |
| 106 | 0.004 | 0.754 | 0.023 | 0.076 | 0.004 | 0.775 | #21 | 0.045 | Not Required | Pass |
| 107 | 0.004 | 0.468 | 0.060 | 0.075 | 0.015 | 0.484 | #21 | 0.074 | Not Required | Pass |
| 108 | 0.001 | 0.051 | 0.052 | 0.044 | 0.006 | 0.094 | #21 | 0.095 | Not Required | Pass |
| 109 | 0.005 | 0.076 | 0.020 | 0.001 | 0.000 | 0.098 | #21 | 0.204 | Not Required | Pass |
| 110 | 0.004 | 0.693 | 0.058 | 0.070 | 0.012 | 0.733 | #21 | 0.080 | Not Required | Pass |
| 111 | 0.001 | 0.060 | 0.053 | 0.048 | 0.006 | 0.093 | #21 | 0.095 | Not Required | Pass |
| 112 | 0.001 | 0.497 | 0.046 | 0.104 | 0.008 | 0.537 | #21 | 0.053 | Not Required | Pass |
| 113 | 0.002 | 0.236 | 0.160 | 0.063 | 0.008 | 0.369 | #21 | 0.286 | Not Required | Pass |
| 114 | 0.003 | 0.233 | 0.159 | 0.059 | 0.008 | 0.365 | #21 | 0.286 | Not Required | Pass |
| 115 | 0.002 | 0.292 | 0.077 | 0.049 | 0.006 | 0.370 | #21 | 0.473 | Not Required | Pass |
| 116 | 0.001 | 0.267 | 0.078 | 0.046 | 0.006 | 0.344 | #21 | 0.473 | Not Required | Pass |
| 201 | 0.313 | 0.480 | 0.003 | 0.011 | 0.000 | 0.682 | #13 | 0.877 | Not Required | Pass |
| 202 | 0.001 | 0.497 | 0.046 | 0.104 | 0.008 | 0.537 | #21 | 0.053 | Not Required | Pass |
| 203 | 0.004 | 0.754 | 0.023 | 0.076 | 0.004 | 0.775 | #21 | 0.045 | Not Required | Pass |
| 204 | 0.004 | 0.693 | 0.058 | 0.070 | 0.012 | 0.733 | #21 | 0.080 | Not Required | Pass |
| 205 | 0.004 | 0.468 | 0.060 | 0.075 | 0.015 | 0.484 | #21 | 0.074 | Not Required | Pass |
| 206 | 0.004 | 0.755 | 0.024 | 0.076 | 0.004 | 0.781 | #21 | 0.045 | Not Required | Pass |
| 207 | 0.004 | 0.468 | 0.069 | 0.075 | 0.017 | 0.486 | #21 | 0.074 | Not Required | Pass |
| 208 | 0.001 | 0.061 | 0.070 | 0.046 | 0.006 | 0.106 | #21 | 0.095 | Not Required | Pass |
| 209 | 0.005 | 0.076 | 0.020 | 0.001 | 0.000 | 0.098 | #21 | 0.204 | Not Required | Pass |
| 210 | 0.004 | 0.701 | 0.066 | 0.070 | 0.014 | 0.746 | #21 | 0.080 | Not Required | Pass |
| 211 | 0.001 | 0.072 | 0.071 | 0.049 | 0.006 | 0.103 | #21 | 0.095 | Not Required | Pass |
| 212 | 0.001 | 0.501 | 0.046 | 0.105 | 0.006 | 0.541 | #21 | 0.053 | Not Required | Pass |
| 213 | 0.002 | 0.236 | 0.160 | 0.063 | 0.008 | 0.369 | #21 | 0.286 | Not Required | Pass |
| 214 | 0.003 | 0.233 | 0.159 | 0.059 | 0.008 | 0.365 | #21 | 0.286 | Not Required | Pass |
| 215 | 0.002 | 0.256 | 0.076 | 0.048 | 0.006 | 0.332 | #21 | 0.473 | Not Required | Pass |
| 216 | 0.002 | 0.226 | 0.077 | 0.044 | 0.006 | 0.304 | #21 | 0.473 | Not Required | Pass |
| 301 | 0.271 | 0.454 | 0.027 | 0.011 | 0.001 | 0.628 | #13 | 0.877 | Not Required | Pass |
| 302 | 0.001 | 0.455 | 0.042 | 0.094 | 0.007 | 0.490 | #21 | 0.053 | Not Required | Pass |
| 303 | 0.004 | 0.681 | 0.029 | 0.069 | 0.006 | 0.710 | #21 | 0.045 | Not Required | Pass |
| 304 | 0.004 | 0.617 | 0.061 | 0.062 | 0.013 | 0.647 | #21 | 0.080 | Not Required | Pass |
| 305 | 0.004 | 0.422 | 0.067 | 0.068 | 0.017 | 0.438 | #21 | 0.074 | Not Required | Pass |
| 306 | 0.003 | 0.623 | 0.013 | 0.062 | 0.002 | 0.638 | #21 | 0.045 | Not Required | Pass |
| 307 | 0.003 | 0.386 | 0.042 | 0.062 | 0.011 | 0.394 | #21 | 0.074 | Not Required | Pass |
| 308 | 0.000 | 0.062 | 0.048 | 0.027 | 0.004 | 0.110 | #21 | Not Required | Not Required | Pass |
| 309 | 0.004 | 0.074 | 0.023 | 0.001 | 0.001 | 0.098 | #21 | 0.204 | Not Required | Pass |

| 309 | 0.004 | 0.074 | 0.025 | 0.001 | 0.001 | 0.050 | #21 | 0.204 | Not Required | Pass |
|-----|-------|-------|-------|-------|-------|-------|-----|--------------|--------------|------|
| 310 | 0.003 | 0.572 | 0.045 | 0.058 | 0.010 | 0.608 | #21 | 0.080 | Not Required | Pass |
| 311 | 0.000 | 0.067 | 0.048 | 0.029 | 0.004 | 0.116 | #21 | Not Required | Not Required | Pass |
| 312 | 0.001 | 0.402 | 0.040 | 0.086 | 0.006 | 0.436 | #21 | 0.035 | Not Required | Pass |
| 313 | 0.002 | 0.207 | 0.149 | 0.060 | 0.008 | 0.290 | #21 | 0.190 | Not Required | Pass |
| 314 | 0.002 | 0.192 | 0.148 | 0.055 | 0.008 | 0.275 | #21 | 0.286 | Not Required | Pass |
| 315 | 0.002 | 0.299 | 0.076 | 0.047 | 0.006 | 0.374 | #21 | 0.473 | Not Required | Pass |
| 316 | 0.001 | 0.274 | 0.077 | 0.042 | 0.006 | 0.352 | #21 | 0.473 | Not Required | Pass |

Definitions

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

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|--------|-------------|--------|--------|-------------|-------|-------|---------------|-------|-------|---------------|--------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1193"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.140</td> <td>11.083</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.483</td> <td>-0.809</td> </tr> <tr> <td>V_z (kip)</td> <td>0.070</td> <td>0.111</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.361</td> <td>0.574</td> </tr> <tr> <td>M_z (kipft)</td> <td>10.497</td> <td>19.209</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 7.140 | 11.083 | V_x (kip) | -0.483 | -0.809 | V_z (kip) | 0.070 | 0.111 | M_x (kipft) | 0.361 | 0.574 | M_z (kipft) | 10.497 | 19.209 | |
| Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Load Component | ASD | LRFD | | | | | | | | | | | | | | | | | | | | | | | | | | |
| P (kip) | 7.140 | 11.083 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.483 | -0.809 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.070 | 0.111 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.361 | 0.574 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 10.497 | 19.209 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.483 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.076911 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(10.497 \text{ kipft}) + ((-0.483 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.6715 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8132 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.07 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.011146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.361 \text{ kipft}) + ((0.07 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.057484 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7967 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8132 \text{ ft}), (1.7967 \text{ ft})]$$

$$L_{e,req} = 4.813 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.813 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9626$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.14 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44625 \text{ kip/ft}^2$$

$$q = 0.44625 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44625 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22312$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.076911 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.6715 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.6715 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.6715 \text{ kipft/ft})) + (4 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3887 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.6715 \text{ kipft/ft})) + (3 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.6715 \text{ kipft/ft})) + (2 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.21628 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.6715 \text{ kipft/ft})) + ((-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.71003 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3887 \text{ ft})}{2}$$

$$p_a = 0.25416 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21628 \text{ kip/ft}^2)}{(0.25416 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.85098$$

p_a - Allowable lateral soil pressure at depth L_e ,

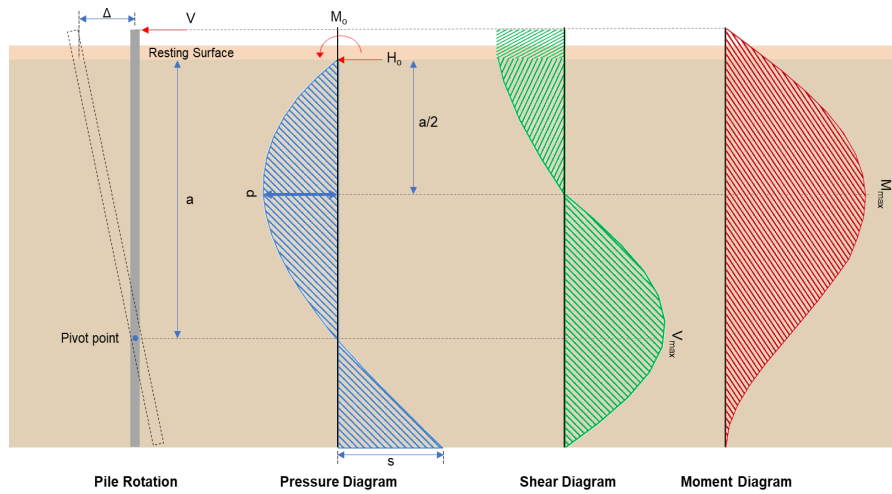
Status: **PASS**
Ratio: **0.850**

| | | |
|--|--|--|
| | $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.71003 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.9467$ | Status: PASS Ratio: 0.950 |
| | <p>Considering z-direction:</p> <p>$H_o = 0.011146 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.057484 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.057484 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.011146 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.057484 \text{ kipft/ft})) + (4 \times (0.011146 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.4969 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.057484 \text{ kipft/ft})) + (3 \times (0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.057484 \text{ kipft/ft})) + (2 \times (0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.016665 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.057484 \text{ kipft/ft})) + ((0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.040968 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.4969 \text{ ft})}{2}$ $p_a = 0.26227 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.016665 \text{ kip/ft}^2)}{(0.26227 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.063541$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ | Status: PASS Ratio: 0.060 |

$$Ratio = \frac{(0.040968 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.054624$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.809 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.12882 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.209 \text{ kipft}) + ((-0.809 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0588 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0588 \text{ kipft/ft})}{(-0.12882 \text{ kip/ft})}$$

$$E = 23.744 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0588 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.12882 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.0588 \text{ kipft/ft})) + (4 \times (-0.12882 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3846 \text{ ft}$$

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

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

$$V_{max} = ((-0.12882 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.744 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3846 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.744 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3846 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.6782 \text{ kip}$$

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

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

$$M_{max} = ((-0.12882 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(23.744 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.744 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (23.744 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 11.459 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.111 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.017675 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.574 \text{ kipft}) + ((0.111 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.091401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.091401 \text{ kipft/ft})}{(0.017675 \text{ kip/ft})}$$

$$E = 5.1712 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.091401 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.017675 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.091401 \text{ kipft/ft})) + (4 \times (0.017675 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4966 \text{ ft}$$

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

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

$$V_{max} = ((0.017675 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4966 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4966 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.17607 \text{ kip}$$

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

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

$$M_{max} = ((0.017675 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.1712 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.40832 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.083 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.228 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.228 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.083 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0041429$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.083 \text{ kip} \rightarrow 11083 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11083 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.96 \text{ kip}$$

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

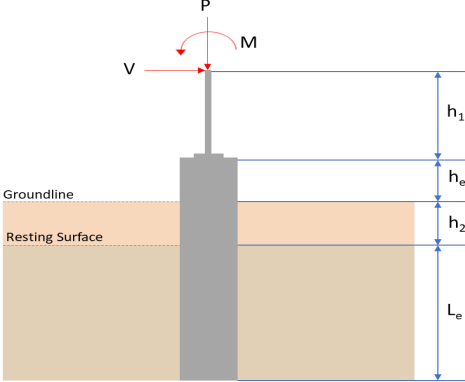
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.96 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.96 \text{ kip}$$

| | | |
|-----------------|---|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.96 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.06 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.6782 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.6782 \text{ kip})}{(111.06 \text{ kip})}$ $\text{Ratio} = 0.042124$ <p>Considering z-direction:</p> <p>$V_{max} = 0.17607 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.17607 \text{ kip})}{(111.06 \text{ kip})}$ $\text{Ratio} = 0.0015854$ | <p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
|------------------|--|---|
| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 11.459 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.459 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.04591$ | <p>Status: PASS Ratio: 0.050</p> |
| | <p>Considering z-direction: $M_{max} = 0.40832 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.40832 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0016359$ | <p>Status: PASS Ratio: 0.000</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|--------|-------------|--------|--------|-------------|--------|--------|---------------|--------|--------|---------------|--------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.241</td> <td>12.822</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.513</td> <td>-0.850</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.007</td> <td>-0.012</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.035</td> <td>-0.063</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.084</td> <td>20.307</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 8.241 | 12.822 | V_x (kip) | -0.513 | -0.850 | V_z (kip) | -0.007 | -0.012 | M_x (kipft) | -0.035 | -0.063 | M_z (kipft) | 11.084 | 20.307 | |
| Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Load Component | ASD | LRFD | | | | | | | | | | | | | | | | | | | | | | | | | | |
| P (kip) | 8.241 | 12.822 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.513 | -0.850 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | -0.007 | -0.012 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | -0.035 | -0.063 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 11.084 | 20.307 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.513 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.081688 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(11.084 \text{ kipft}) + ((-0.513 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.765 \text{ kipft/ft}$$

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:

$L_{e,x}$ = 4.8939 ft - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.007 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0011146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((-0.007 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0055732 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z}$ = 0.73475 ft - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8939 \text{ ft}), (0.73475 \text{ ft})]$$

$$L_{e,req} = 4.894 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.894 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9788$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.241 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51506 \text{ kip/ft}^2$$

$$q = 0.01500 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51506 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25753$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.081688 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.765 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.765 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.765 \text{ kipft/ft})) + (4 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.389 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.765 \text{ kipft/ft})) + (3 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.765 \text{ kipft/ft})) + (2 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.22806 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.765 \text{ kipft/ft})) + ((-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74916 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.389 \text{ ft})}{2}$$

$$p_a = 0.25418 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22806 \text{ kip/ft}^2)}{(0.25418 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89725$$

p_s - Allowable lateral soil pressure at depth L_e ,

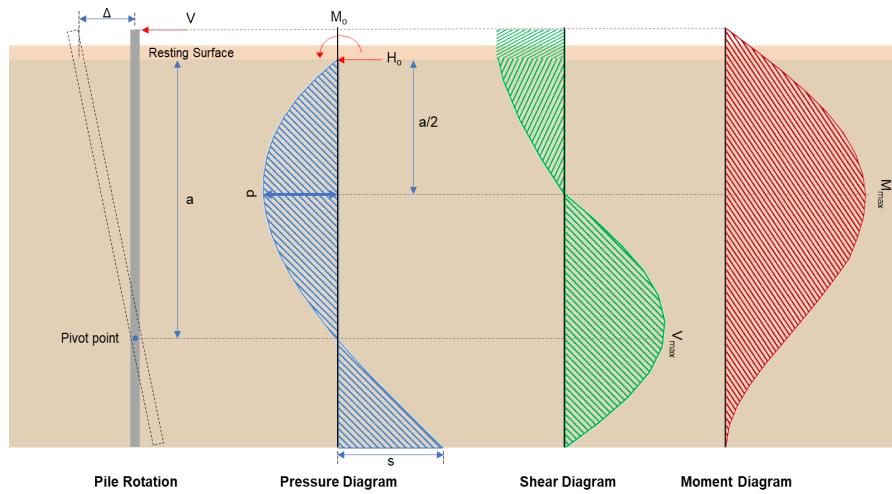
Status: **PASS**
Ratio: **0.900**

| | | |
|--|---|--|
| | $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74916 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99888$ | Status: PASS Ratio: 1.000 |
| | <p>Considering z-direction:</p> <p>$H_o = -0.0011146 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0055732 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0055732 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.0011146 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.0055732 \text{ kipft/ft})) + (4 \times (-0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0055732 \text{ kipft/ft})) + (3 \times (-0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.0055732 \text{ kipft/ft})) + (2 \times (-0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.0001672 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0055732 \text{ kipft/ft})) + ((-0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.0013376 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5 \text{ ft})}{2}$ $p_a = 0.2625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0001672 \text{ kip/ft}^2)}{(0.2625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.00063694$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ | Status: PASS Ratio: 0.000 |

$$Ratio = \frac{(0.0013376 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.0017834$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.85 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.307 \text{ kipft}) + ((-0.85 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2336 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.2336 \text{ kipft/ft})}{(-0.13535 \text{ kip/ft})}$$

$$E = 23.891 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2336 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.13535 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.2336 \text{ kipft/ft})) + (4 \times (-0.13535 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3844 \text{ ft}$$

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

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

$$V_{max} = ((-0.13535 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3844 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3844 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9435 \text{ kip}$$

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

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

$$M_{max} = ((-0.13535 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(23.891 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 12.11 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.012 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.0019108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.063 \text{ kipft}) + ((-0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010032 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010032 \text{ kipft/ft})}{(-0.0019108 \text{ kip/ft})}$$

$$E = 5.25 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010032 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.0019108 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.010032 \text{ kipft/ft})) + (4 \times (-0.0019108 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4951 \text{ ft}$$

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

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

$$V_{max} = ((-0.0019108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4951 \text{ ft})}{(5 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4951 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019248 \text{ kip}$$

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

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

$$M_{max} = ((-0.0019108 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.25 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 + \left[\left(\frac{3 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right] \right]$$

$$M_{max} = 0.044673 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.822 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.17 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.17 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2 ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.822 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0047929$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1 The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,
 $V_{c,max}$ - Max shear strength of concrete

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.822 \text{ kip} \rightarrow 12822 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12822 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

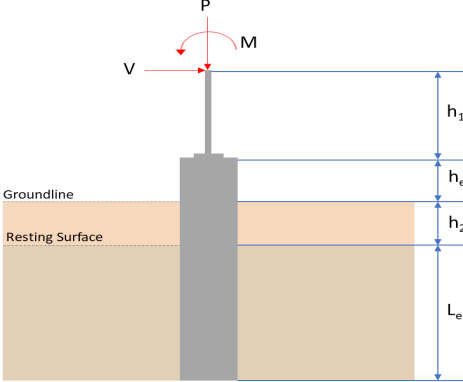
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

| | | |
|-----------------|--|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.21 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9435 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9435 \text{ kip})}{(111.21 \text{ kip})}$ $\text{Ratio} = 0.044453$ <p>Considering z-direction:</p> <p>$V_{max} = 0.019248 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.019248 \text{ kip})}{(111.21 \text{ kip})}$ $\text{Ratio} = 0.00017308$ | <p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
|------------------|---|---|
| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.11 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.11 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.048519$ | <p>Status: PASS Ratio: 0.050</p> |
| | <p>Considering z-direction: $M_{max} = 0.044673 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.044673 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00017898$ | <p>Status: PASS Ratio: 0.000</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|--------|-------------|--------|--------|-------------|-------|-------|---------------|-------|-------|---------------|--------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>8.241</td> <td>12.822</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.513</td> <td>-0.850</td> </tr> <tr> <td>V_z (kip)</td> <td>0.007</td> <td>0.012</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.035</td> <td>0.063</td> </tr> <tr> <td>M_z (kipft)</td> <td>11.084</td> <td>20.307</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_n) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 8.241 | 12.822 | V_x (kip) | -0.513 | -0.850 | V_z (kip) | 0.007 | 0.012 | M_x (kipft) | 0.035 | 0.063 | M_z (kipft) | 11.084 | 20.307 | |
| Layer | Label | Allowable Bearing Pressure (q_n) (psf) | Allowable Lateral Pressure (R) (psf/ft) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Load Component | ASD | LRFD | | | | | | | | | | | | | | | | | | | | | | | | | | |
| P (kip) | 8.241 | 12.822 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.513 | -0.850 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.007 | 0.012 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.035 | 0.063 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 11.084 | 20.307 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.513 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.081688 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(11.084 \text{ kipft}) + ((-0.513 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.765 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8939 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.007 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0011146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.035 \text{ kipft}) + ((0.007 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.0055732 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 0.79286 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8939 \text{ ft}), (0.79286 \text{ ft})]$$

$$L_{e,req} = 4.894 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.894 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9788$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(8.241 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.51506 \text{ kip/ft}^2$$

$$q = 0.01500 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.51506 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.25753$$

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

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.081688 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.765 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.765 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.765 \text{ kipft/ft})) + (4 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.389 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.765 \text{ kipft/ft})) + (3 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.765 \text{ kipft/ft})) + (2 \times (-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.22806 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.765 \text{ kipft/ft})) + ((-0.081688 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.74916 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.389 \text{ ft})}{2}$$

$$p_a = 0.25418 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.22806 \text{ kip/ft}^2)}{(0.25418 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.89725$$

p_a - Allowable lateral soil pressure at depth L_e ,

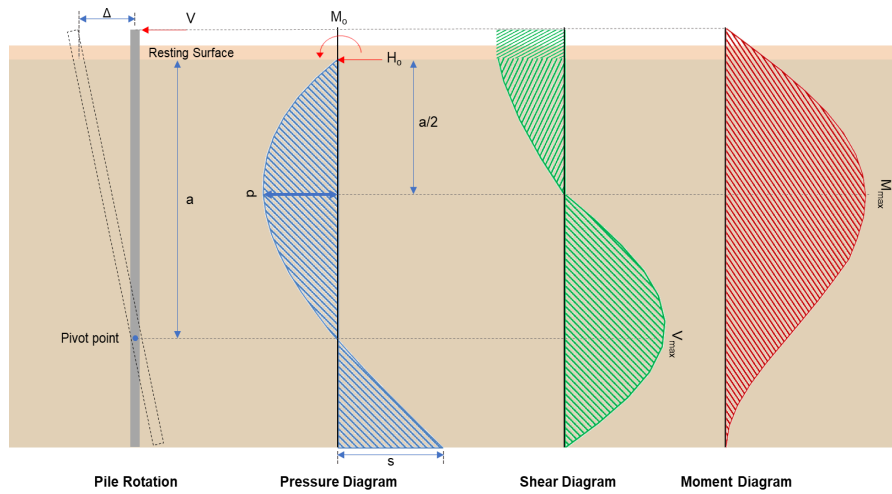
Status: **PASS**
Ratio: **0.900**

| | | |
|--|---|--|
| | $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.74916 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.99888$ | Status: PASS Ratio: 1.000 |
| | <p>Considering z-direction:</p> <p>$H_o = 0.0011146 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.0055732 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.0055732 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.0011146 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.0055732 \text{ kipft/ft})) + (4 \times (0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))}$ $a = 3.5 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 \times [(4 \times (0.0055732 \text{ kipft/ft})) + (3 \times (0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (0.0055732 \text{ kipft/ft})) + (2 \times (0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]}$ $p = 0.0016385 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 \times [(2 \times (0.0055732 \text{ kipft/ft})) + ((0.0011146 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$ $s = 0.0040127 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(3.5 \text{ ft})}{2}$ $p_a = 0.2625 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0016385 \text{ kip/ft}^2)}{(0.2625 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.006242$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$ $p_s = 0.75 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ | Status: PASS Ratio: 0.010 |

$$Ratio = \frac{(0.0040127 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.0053503$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.85 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.13535 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(20.307 \text{ kipft}) + ((-0.85 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.2336 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.2336 \text{ kipft/ft})}{(-0.13535 \text{ kip/ft})}$$

$$E = 23.891 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2336 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.13535 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.2336 \text{ kipft/ft})) + (4 \times (-0.13535 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3844 \text{ ft}$$

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

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

$$V_{max} = ((-0.13535 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3844 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3844 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.9435 \text{ kip}$$

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

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

$$M_{max} = ((-0.13535 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(23.891 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.891 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3844 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 12.11 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.012 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.0019108 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.063 \text{ kipft}) + ((0.012 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.010032 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.010032 \text{ kipft/ft})}{(0.0019108 \text{ kip/ft})}$$

$$E = 5.25 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.010032 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (0.0019108 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.010032 \text{ kipft/ft})) + (4 \times (0.0019108 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4951 \text{ ft}$$

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

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

$$V_{max} = ((0.0019108 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4951 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4951 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.019248 \text{ kip}$$

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

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

$$M_{max} = ((0.0019108 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.25 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.25 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4951 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.044673 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(12.822 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.17 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.17 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(12.822 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0047929$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 12.822 \text{ kip} \rightarrow 12822 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(12822 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 120.19 \text{ kip}$$

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

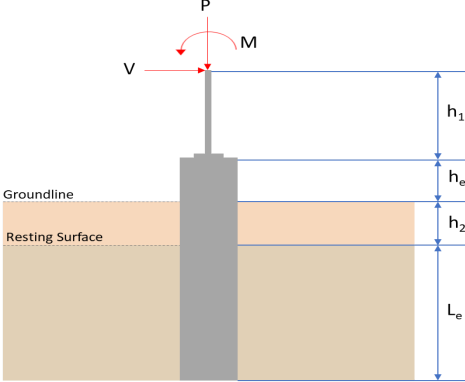
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (120.19 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 120.19 \text{ kip}$$

| | | |
|-----------------|--|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((120.19 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.21 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.9435 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.9435 \text{ kip})}{(111.21 \text{ kip})}$ $\text{Ratio} = 0.044453$ <p>Considering z-direction:</p> <p>$V_{max} = 0.019248 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.019248 \text{ kip})}{(111.21 \text{ kip})}$ $\text{Ratio} = 0.00017308$ | <p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
|------------------|---|---|
| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 12.11 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(12.11 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.048519$ | <p>Status: PASS Ratio: 0.050</p> |
| | <p>Considering z-direction: $M_{max} = 0.044673 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.044673 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.00017898$ | <p>Status: PASS Ratio: 0.000</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|--------|-------------|--------|--------|-------------|--------|--------|---------------|--------|--------|---------------|--------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry</p> <p>Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 5$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>7.140</td> <td>11.083</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.483</td> <td>-0.809</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.070</td> <td>-0.111</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.361</td> <td>-0.574</td> </tr> <tr> <td>M_z (kipft)</td> <td>10.497</td> <td>19.210</td> </tr> </tbody> </table> <p>Material Properties</p> <p>$f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 7.140 | 11.083 | V_x (kip) | -0.483 | -0.809 | V_z (kip) | -0.070 | -0.111 | M_x (kipft) | -0.361 | -0.574 | M_z (kipft) | 10.497 | 19.210 | |
| Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Load Component | ASD | LRFD | | | | | | | | | | | | | | | | | | | | | | | | | | |
| P (kip) | 7.140 | 11.083 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.483 | -0.809 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | -0.070 | -0.111 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | -0.361 | -0.574 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 10.497 | 19.210 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD)</p> <p>H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction:</p> <p>H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-0.483 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.076911 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(10.497 \text{ kipft}) + ((-0.483 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.6715 \text{ kipft/ft}$$

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:

$L_{e,x} = 4.8132 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.07 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.011146 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.361 \text{ kipft}) + ((-0.07 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.057484 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.5291 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.8132 \text{ ft}), (1.5291 \text{ ft})]$$

$$L_{e,req} = 4.813 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 5 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.813 \text{ ft})}{(5 \text{ ft})}$$

$$\text{Ratio} = 0.9626$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(7.14 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.44625 \text{ kip/ft}^2$$

$$q = 0.44625 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.44625 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.22312$$

Status: **PASS**
Ratio: **0.220**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.076911 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 1.6715 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.6715 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (1.6715 \text{ kipft/ft})) + (4 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3887 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (1.6715 \text{ kipft/ft})) + (3 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 \times [(3 \times (1.6715 \text{ kipft/ft})) + (2 \times (-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.21628 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (1.6715 \text{ kipft/ft})) + ((-0.076911 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.71003 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.3887 \text{ ft})}{2}$$

$$p_a = 0.25416 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.21628 \text{ kip/ft}^2)}{(0.25416 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.85098$$

p_a - Allowable lateral soil pressure at depth L_e ,

Status: **PASS**
Ratio: **0.850**

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.71003 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.9467$$

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

Considering z-direction:

$H_o = -0.011146 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.057484 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.057484 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.011146 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.057484 \text{ kipft/ft})) + (4 \times (-0.011146 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4969 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.057484 \text{ kipft/ft})) + (3 \times (-0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]^2}{(5 \text{ ft})^2 [(3 \times (0.057484 \text{ kipft/ft})) + (2 \times (-0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]}$$

$$p = 0.0019362 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 [(2 \times (0.057484 \text{ kipft/ft})) + ((-0.011146 \text{ kip/ft}) \times (5 \text{ ft}))]}{(5 \text{ ft})^2}$$

$$s = 0.014217 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(3.4969 \text{ ft})}{2}$$

$$p_a = 0.26227 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.0019362 \text{ kip/ft}^2)}{(0.26227 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.0073826$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (5 \text{ ft})$$

$$p_s = 0.75 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

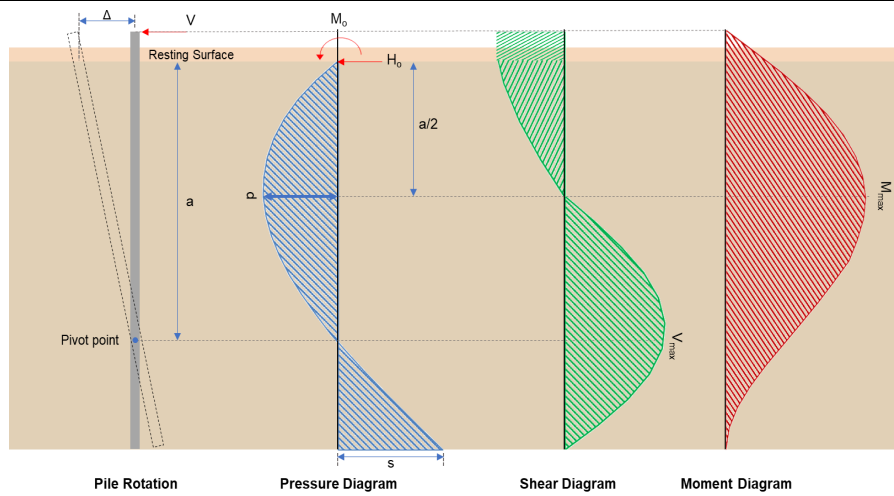
$$\text{Ratio} = \frac{s}{p_s}$$

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

$$Ratio = \frac{(0.014217 \text{ kip/ft}^2)}{(0.75 \text{ kip/ft}^2)}$$

$$Ratio = 0.018955$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.809 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.12882 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(19.21 \text{ kipft}) + ((-0.809 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.0589 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(3.0589 \text{ kipft/ft})}{(-0.12882 \text{ kip/ft})}$$

$$E = 23.745 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.0589 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.12882 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (3.0589 \text{ kipft/ft})) + (4 \times (-0.12882 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.3846 \text{ ft}$$

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

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

$$V_{max} = ((-0.12882 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.745 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3846 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (23.745 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3846 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.6784 \text{ kip}$$

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

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

$$M_{max} = ((-0.12882 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(23.745 \text{ ft})}{(5 \text{ ft})} + \frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.745 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.745 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.3846 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 11.46 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.111 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.017675 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.574 \text{ kipft}) + ((-0.111 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.091401 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.091401 \text{ kipft/ft})}{(-0.017675 \text{ kip/ft})}$$

$$E = 5.1712 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.091401 \text{ kipft/ft}) \times (5 \text{ ft})) + (3 \times (-0.017675 \text{ kip/ft}) \times (5 \text{ ft})^2)}{(6 \times (0.091401 \text{ kipft/ft})) + (4 \times (-0.017675 \text{ kip/ft}) \times (5 \text{ ft}))}$$

$$a = 3.4966 \text{ ft}$$

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

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

$$V_{max} = ((-0.017675 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4966 \text{ ft})}{(5 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4966 \text{ ft})}{(5 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.17607 \text{ kip}$$

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

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

$$M_{max} = ((-0.017675 \text{ kip/ft}) \times (48 \text{ in}) \times (5 \text{ ft})) \times \left[\left(\frac{(5.1712 \text{ ft})}{(5 \text{ ft})} + \frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 3 \right) \times \left(\frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.1712 \text{ ft})}{(5 \text{ ft})} + 2 \right) \times \left(\frac{(3.4966 \text{ ft})}{2 \times (5 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.40832 \text{ kipft}$$

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,

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(11.083 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.228 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.228 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$$

$$\phi P_N = 2675.2 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(11.083 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.0041429$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 11.083 \text{ kip} \rightarrow 11083 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(11083 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.96 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.96 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.96 \text{ kip}$$

| | | |
|-----------------|--|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yties} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.96 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 111.06 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.6784 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(4.6784 \text{ kip})}{(111.06 \text{ kip})}$ $\text{Ratio} = 0.042126$ <p>Considering z-direction:</p> <p>$V_{max} = 0.17607 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.17607 \text{ kip})}{(111.06 \text{ kip})}$ $\text{Ratio} = 0.0015854$ | <p>Status: PASS Ratio: 0.040</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
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| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ksi}} \times 18432.001 \text{in}^3$ $\phi M_{n,1} = 249.600 \text{kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ksi}) \times (18432 \text{in}^3)$ $\phi M_{n,2} = 2121.6 \text{kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{kipft}), (2121.6 \text{kipft})]$ $\phi M_n = 249.6 \text{kipft}$ <p>Considering x-direction: $M_{max} = 11.46 \text{kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(11.46 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.045912$ | <p>Status: PASS Ratio: 0.050</p> |
| | <p>Considering z-direction: $M_{max} = 0.40832 \text{kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.40832 \text{kipft})}{(249.6 \text{kipft})}$ $\text{Ratio} = 0.0016359$ | <p>Status: PASS Ratio: 0.000</p> |