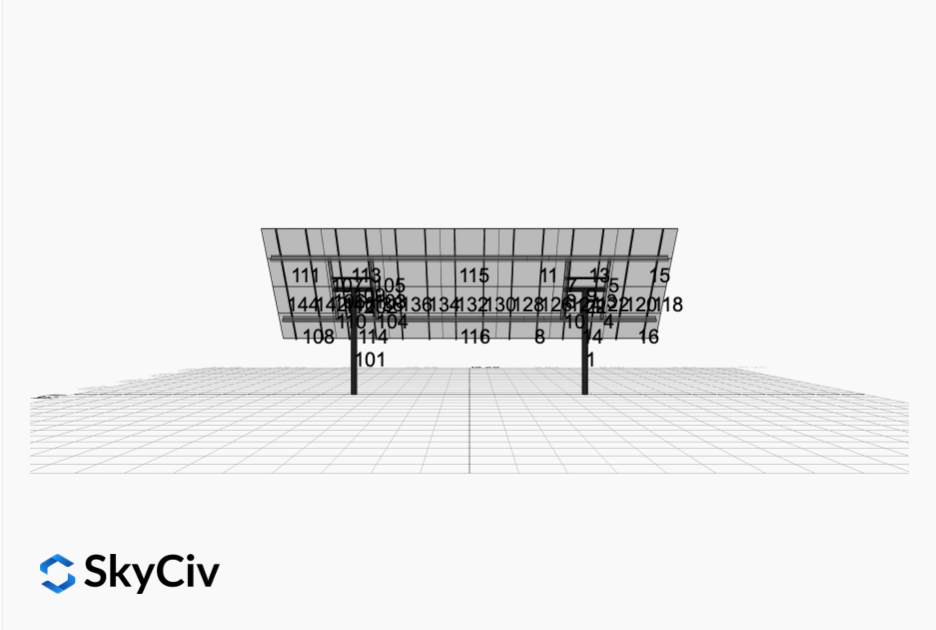


Project Details



Project Name: Wallewein Rd
Location: 108 Wallewein Rd, Sunburst, MT 59482, USA
Unique ID: 2P-22.5-6TOP-SD-45-L-4Hx7W-LI94
Dealer: _____

Date: Wed Feb 05 2025
Number of Modules: 28
Number of Poles: 2
Date Sold: _____



| | |
|----------------------|----------|
| Array Dimensions N/S | 13.50 ft |
| Array Dimensions E/W | 38.50 ft |
| Winter Tilt Angle | 50 |
| Front Edge Clearance | 5 ft |

MT Solar Bill of Materials (2P-22.5-6TOP-SD-45-L-4Hx7W-LI94)

| Part | Short Description | BOM Qty |
|--------------------|-----------------------|---------|
| MTS-PC-6 | 6IN Pole Cap Assembly | 2 |
| MTS-HF-SD | H-Frame Assembly-SD | 2 |
| MTS-SD-Wing-45 | 45IN SD Wing | 4 |
| MTS-SD-Splice-90 | 90IN SD Splice | 4 |
| MTS-CLAMP-HOOK-4PK | Hook Clamp | 7 |

Rail Bill of Materials

| Part | Qty |
|------------------|-----|
| Rails (160in) | 14 |
| Rail Attachment | 28 |
| Module Mid Clamp | 42 |
| Module End Clamp | 28 |
| Ground Lug | 7 |

Site Details:



Site Address: 108 Wallewein Rd, Sunburst, MT 59482, USA

Array Specification

| | |
|------------------------------------|-----------|
| Duty Classification: | SD |
| Module Width: | 40.00 in |
| Module Length: | 65.00in |
| Number of Rows: | 4 |
| Number of Columns: | 7 |
| Total Number of Modules: | 28 |
| Winter Tilt Angle: | 50 |
| Front Edge Clearance: | 5 |
| Total Array Height at Tilt: | 15.34 ft |
| Total Frame Length: | 37.50 ft |
| Frame Weight: | 2094 lbs |
| Array Dimensions N/S: | 13.50 ft |
| Array Dimensions E/W: | 38.50 ft |
| Rail Length: | 162.00 in |
| Rail Spacing: | 2.75 ft |

Support Specifications

| | |
|---------------------------------|-----------------|
| Pole Size: | 6in Pipe Sch 80 |
| Pole Length above Grade: | 10.17 ft |
| Number of Poles: | 2 |
| Pole Spacing: | 22.5 ft |

Foundation Specifications

| | |
|--|------------------------------------|
| Foundation Type: | Square |
| Foundation Dimensions: | 48 x 48 in |
| Foundation Depth (below grade): | Pile 1: 6.25 ft Pile 2: 6.25 ft |
| Foundation Volume: | 7.407 y ³ |

Site Info

| | |
|-----------------------------|---|
| Risk Category: | I |
| Exposure: | C |
| Soil Classification: | sand |
| Site Location: | 108 Wallewein Rd, Sunburst, MT 59482, USA |
| Wind Speed: | 105 mph |
| Snow Load: | 35 psf |

Design Disclaimer

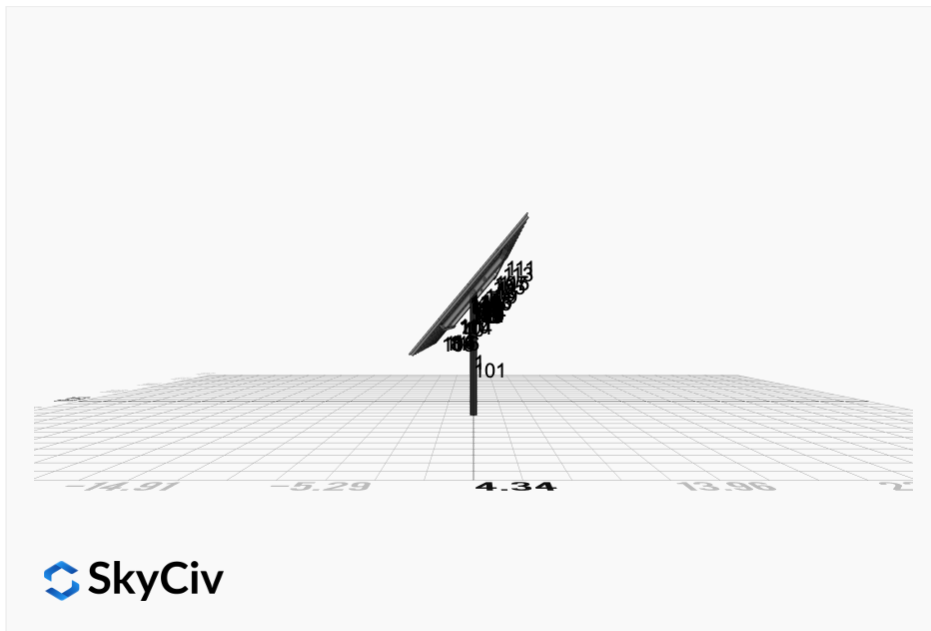
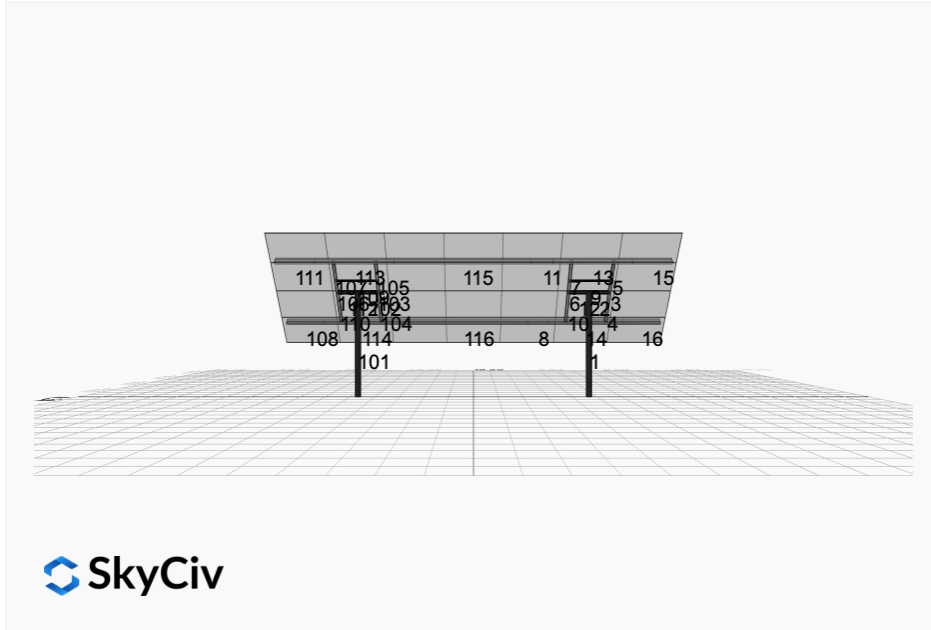
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

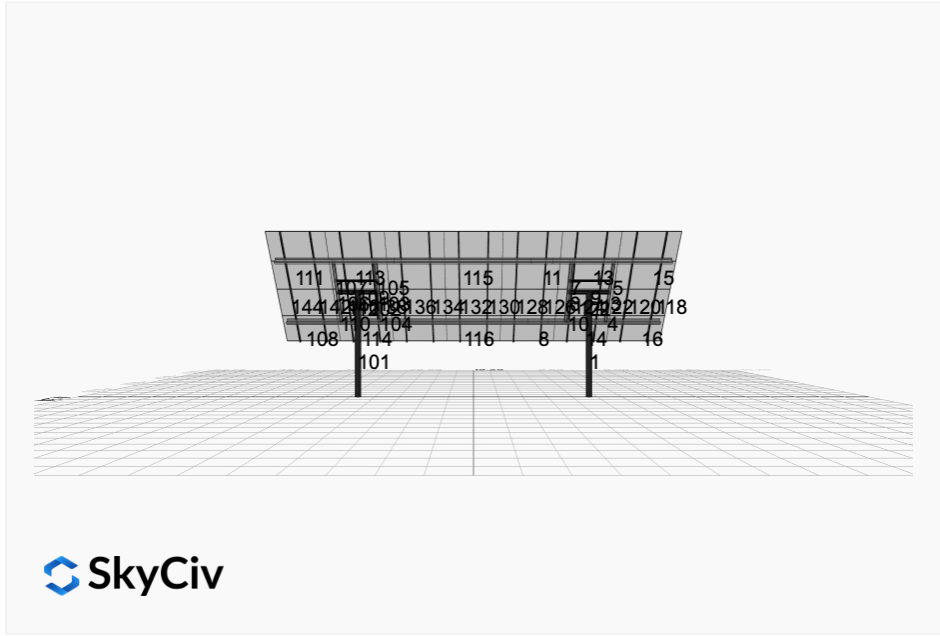
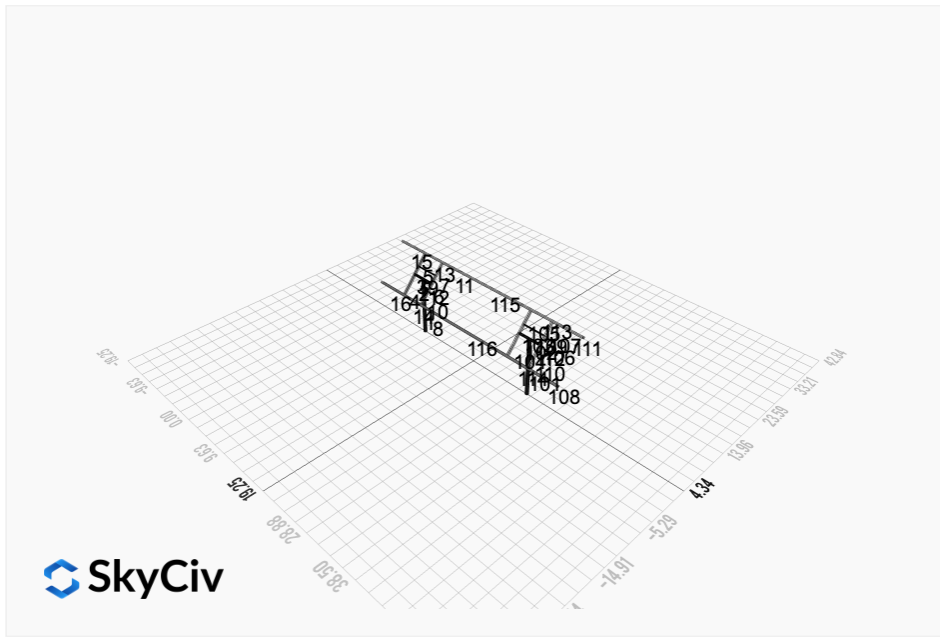
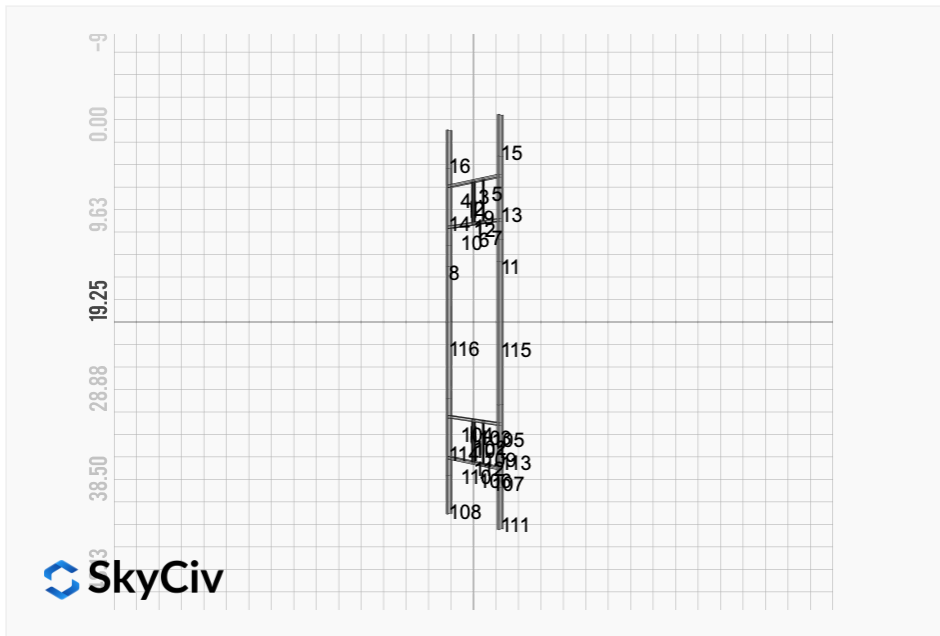
AutoDesigner Input

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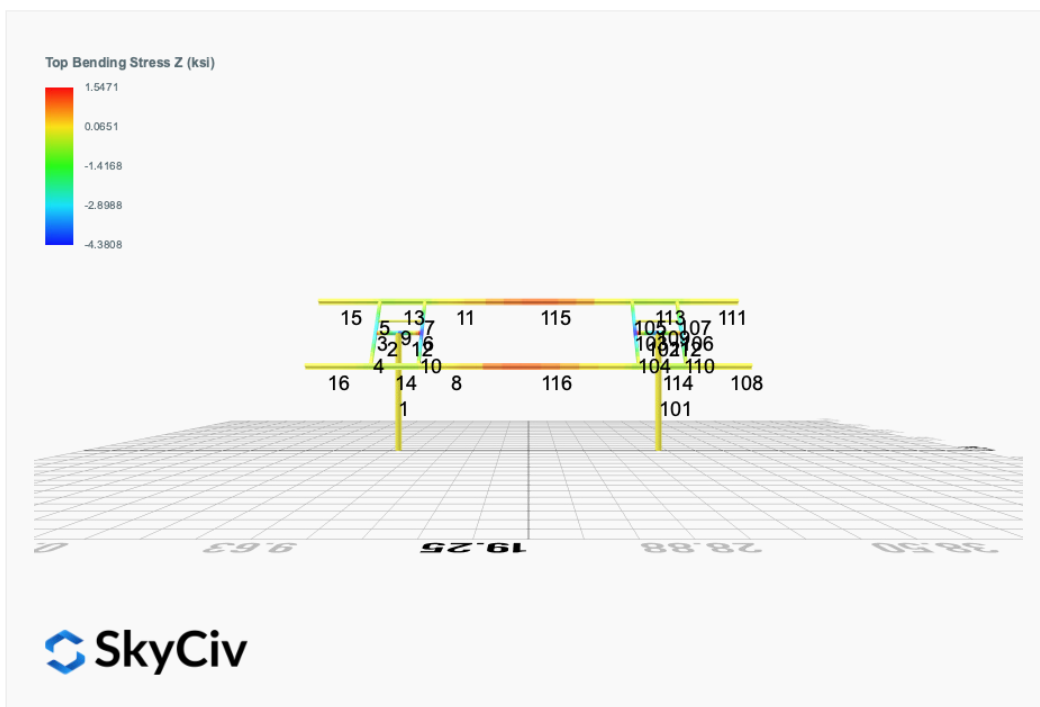
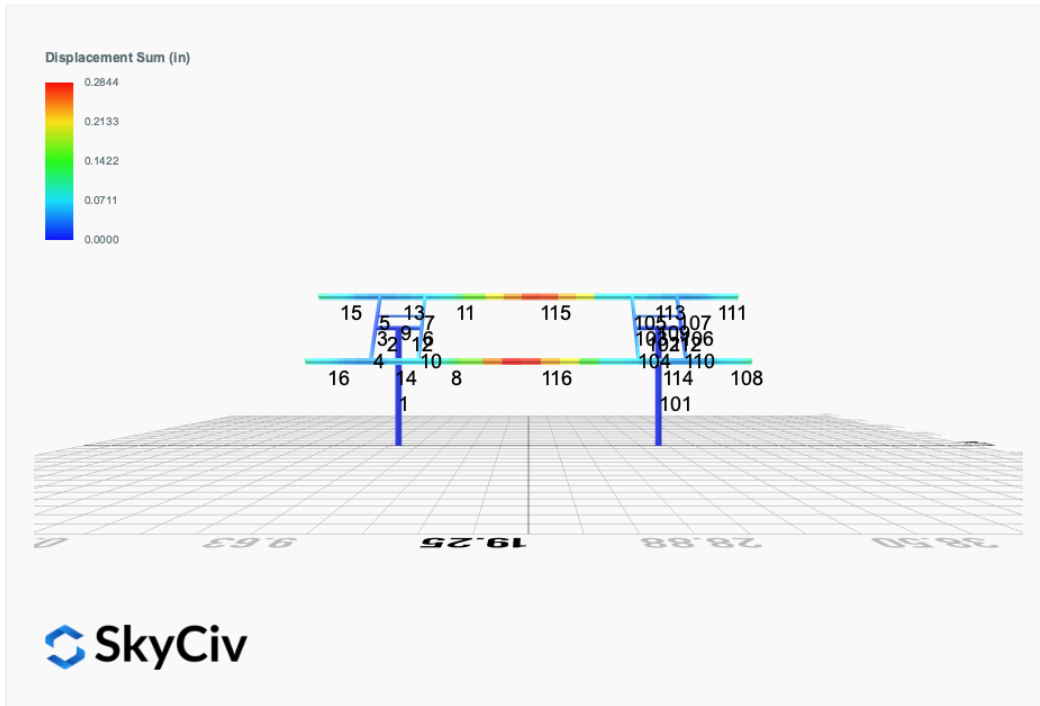
Design Notes:

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

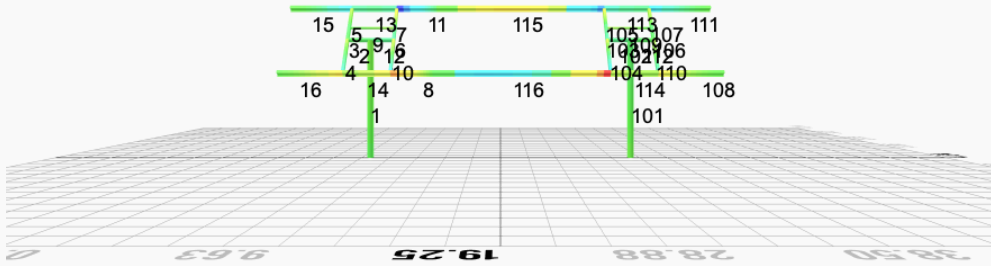
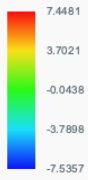




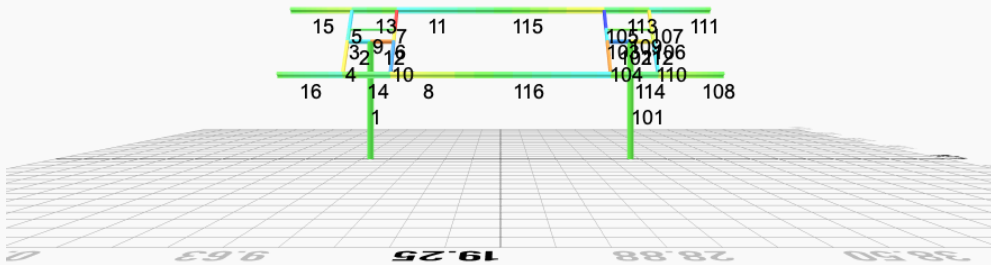
FEM Results (Envelope Worst Case for each member)

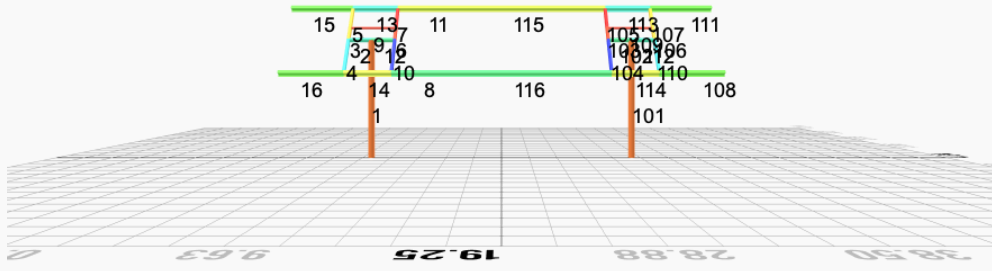


Top Bending Stress Y (ksi)



Shear Stress Y (ksi)





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

ASD Load Combination Results

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 1. D | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 2. D + L | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 3.2830 | 0.1441 | 0.4385 | -0.2150 | 0.0195 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 2.9699 | 0.1282 | 0.3901 | -0.1913 | 0.0188 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 5b. D + 0.7E | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0000 | 2.9699 | 0.1282 | 0.3901 | -0.1913 | 0.0188 |
| ULS: 8. 0.6D + 0.7E | 0.0000 | 1.2183 | 0.0484 | 0.1471 | -0.0721 | 0.0100 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -2.4183 | 4.0597 | 0.2523 | 0.7411 | -0.7961 | 24.9707 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 2.4183 | 0.0014 | -0.0902 | -0.2470 | 0.5538 | -24.2331 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.8137 | 4.4917 | 0.2570 | 0.7621 | -0.6982 | 18.7344 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | 0.0000 | 2.9699 | 0.1282 | 0.3901 | -0.1913 | 0.0188 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.8137 | 1.4480 | 0.0002 | 0.0210 | 0.3141 | -18.1684 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.0000 | 2.9699 | 0.1282 | 0.3901 | -0.1913 | 0.0188 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.8137 | 3.5524 | 0.2094 | 0.6171 | -0.6271 | 18.7322 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.8137 | 0.5087 | -0.0475 | -0.1240 | 0.3853 | -18.1707 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.0000 | 2.0305 | 0.0806 | 0.2452 | -0.1202 | 0.0166 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -2.4183 | 3.2475 | 0.2200 | 0.6430 | -0.7480 | 24.9641 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | 0.0000 | 1.2183 | 0.0484 | 0.1471 | -0.0721 | 0.0100 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 2.4183 | -0.8109 | -0.1224 | -0.3451 | 0.6018 | -24.2397 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.0000 | 1.2183 | 0.0484 | 0.1471 | -0.0721 | 0.0100 |

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 | 6.4448 |
| Shear X | -4.0305 |
| Shear Z | 0.4150 |
| Moment X | 1.2197 |
| Moment Y (Twist) | 1.3170 |
| Moment Z | 42.2203 |

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 | 4.4917 |
| Shear X | -2.4183 |
| Shear Z | 0.2570 |
| Moment X | 0.7621 |
| Moment Y (Twist) | 0.7961 |
| Moment Z | 24.9707 |

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 | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 2. D + L | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 3.2830 | -0.1441 | -0.4385 | 0.2151 | 0.0196 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 2.9698 | -0.1282 | -0.3902 | 0.1914 | 0.0188 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 5b. D + 0.7E | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0000 | 2.9698 | -0.1282 | -0.3902 | 0.1914 | 0.0188 |
| ULS: 8. 0.6D + 0.7E | -0.0000 | 1.2183 | -0.0484 | -0.1471 | 0.0721 | 0.0100 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -2.4183 | 4.0597 | -0.2523 | -0.7412 | 0.7961 | 24.9708 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 2.4183 | 0.0014 | 0.0902 | 0.2470 | -0.5537 | -24.2330 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.8137 | 4.4917 | -0.2570 | -0.7622 | 0.6983 | 18.7345 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.0000 | 2.9698 | -0.1282 | -0.3902 | 0.1914 | 0.0188 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.8137 | 1.4480 | -0.0002 | -0.0210 | -0.3141 | -18.1684 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | -0.0000 | 2.9698 | -0.1282 | -0.3902 | 0.1914 | 0.0188 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.8137 | 3.5524 | -0.2094 | -0.6172 | 0.6271 | 18.7322 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.8137 | 0.5087 | 0.0475 | 0.1239 | -0.3853 | -18.1706 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | -0.0000 | 2.0305 | -0.0806 | -0.2452 | 0.1202 | 0.0166 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -2.4183 | 3.2475 | -0.2200 | -0.6431 | 0.7480 | 24.9641 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.0000 | 1.2183 | -0.0484 | -0.1471 | 0.0721 | 0.0100 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 2.4183 | -0.8109 | 0.1224 | 0.3451 | -0.6018 | -24.2397 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | -0.0000 | 1.2183 | -0.0484 | -0.1471 | 0.0721 | 0.0100 |

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 | 6.4448 |
| Shear X | -4.0305 |
| Shear Z | -0.4150 |
| Moment X | -1.2202 |
| Moment Y (Twist) | 1.3172 |
| Moment Z | 42.2212 |

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 | 4.4917 |
| Shear X | -2.4183 |
| Shear Z | -0.2570 |
| Moment X | -0.7622 |
| Moment Y (Twist) | 0.7961 |
| Moment Z | 24.9708 |

Project Details

Design Code: AISC 360-16 LRFD
 Provision: LRFD
 Country: United States
 User Name: sales@mtsolar.us
 Project Name: Wallewein Rd
 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) | | | | | |
|----|-----------------|--------|------------|--|--|--|--|--|
| 1 | 2in Pipe Sch 40 | 2.38 | 0.15 | | | | | |
| 4 | 4in Pipe Sch 40 | 4.50 | 0.24 | | | | | |
| 8 | 6in Pipe Sch 80 | 6.63 | 0.43 | | | | | |

| ID | Name | d (in) | b (in) | t_w (in) | t_b (in) | r (in) | | |
|----|------------|--------|--------|------------|------------|--------|--|--|
| 15 | HSS5x3x1/8 | 5.00 | 3.00 | 0.12 | 0.12 | 0.12 | | |

| ID | Name | d (in) | t_w (in) | b_t (in) | b_b (in) | t_t (in) | t_b (in) | r (in) |
|----|------|--------|------------|------------|------------|------------|------------|--------|
| 18 | W6x9 | 5.90 | 0.17 | 3.94 | 3.94 | 0.21 | 0.21 | 0.25 |

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

| | | | | | | | | |
|-----|----|------|------|-------|--|-----|-----|---|
| 113 | 18 | 4.88 | 4.00 | 7.50 | 1.07,1.07,1.07,1.07,1.07,1.07,1.08,1.07,1.09,1.07,1.08,1.07,1.09,1.07,1.08,1.07,1.63,1.07,1.08,1.07,1.10,1.07,1.08,1.07,1.09,1.07 | 300 | 200 | 1 |
| 114 | 18 | 4.88 | 4.00 | 7.50 | 1.07,1.07,1.07,1.07,1.07,1.07,1.08,1.07,1.09,1.07,1.08,1.07,1.09,1.07,1.08,1.07,1.69,1.07,1.08,1.07,1.10,1.07,1.08,1.07,1.09,1.07 | 300 | 200 | 1 |
| 115 | 18 | 8.42 | 8.42 | 12.95 | 1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.12,1.12,1.07,1.12,1.12,1.12,1.11,1.12,1.12,1.12,1.11,1.12 | 300 | 200 | 1 |
| 116 | 18 | 8.42 | 8.42 | 12.95 | 1.12,1.12,1.12,1.12,1.12,1.12,1.11,1.12,1.12,1.11,1.12,1.11,1.12,1.11,1.12,1.11,1.12,1.12,1.12,1.07,1.12,1.12,1.12,1.11,1.12,1.11,1.12,1.11,1.12 | 300 | 200 | 1 |

Member Design Capacity

| Member ID | $\Phi_t P_n$ (kip) | $\Phi_c P_n$ (kip) | $\Phi_b M_{zn}$ (k-ft) | $\Phi_b M_{yn}$ (k-ft) | $\Phi_v V_{yn}$ (kip) | $\Phi_v V_{zn}$ (kip) |
|-----------|--------------------|--------------------|------------------------|------------------------|-----------------------|-----------------------|
| 1 | 378.22 | 139.25 | 62.23 | 62.23 | 113.47 | 113.47 |
| 2 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 3 | 79.65 | 74.02 | 10.99 | 6.26 | 29.14 | 16.61 |
| 4 | 79.65 | 72.01 | 10.99 | 6.26 | 29.14 | 16.61 |
| 5 | 79.65 | 73.44 | 10.99 | 6.26 | 29.14 | 16.61 |
| 6 | 79.65 | 74.02 | 10.99 | 6.26 | 29.14 | 16.61 |
| 7 | 79.65 | 73.44 | 10.99 | 6.26 | 29.14 | 16.61 |
| 8 | 120.60 | 115.40 | 23.36 | 6.45 | 30.09 | 45.74 |
| 9 | 48.35 | 43.11 | 2.85 | 2.85 | 14.51 | 14.51 |
| 10 | 79.65 | 72.01 | 10.99 | 6.26 | 29.14 | 16.61 |
| 11 | 120.60 | 115.40 | 23.36 | 6.45 | 30.09 | 45.74 |
| 12 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 13 | 120.60 | 84.03 | 18.91 | 6.45 | 30.09 | 45.74 |
| 14 | 120.60 | 84.03 | 18.83 | 6.45 | 30.09 | 45.74 |
| 15 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 16 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 101 | 378.22 | 139.25 | 62.23 | 62.23 | 113.47 | 113.47 |
| 102 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 103 | 79.65 | 74.02 | 10.99 | 6.26 | 29.14 | 16.61 |
| 104 | 79.65 | 72.01 | 10.99 | 6.26 | 29.14 | 16.61 |
| 105 | 79.65 | 73.44 | 10.99 | 6.26 | 29.14 | 16.61 |
| 106 | 79.65 | 74.02 | 10.99 | 6.26 | 29.14 | 16.61 |
| 107 | 79.65 | 73.44 | 10.99 | 6.26 | 29.14 | 16.61 |
| 108 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 109 | 48.35 | 43.11 | 2.85 | 2.85 | 14.51 | 14.51 |
| 110 | 79.65 | 72.01 | 10.99 | 6.26 | 29.14 | 16.61 |
| 111 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 112 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 113 | 120.60 | 84.03 | 18.91 | 6.45 | 30.09 | 45.74 |
| 114 | 120.60 | 84.03 | 18.83 | 6.45 | 30.09 | 45.74 |
| 115 | 120.60 | 48.60 | 10.56 | 6.45 | 30.09 | 45.74 |
| 116 | 120.60 | 48.60 | 10.55 | 6.45 | 30.09 | 45.74 |

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 | 0.046 | 0.678 | 0.048 | 0.036 | 0.004 | 0.721 | #13 | 0.584 | Not Required | Pass |
| 2 | 0.004 | 0.218 | 0.209 | 0.055 | 0.043 | 0.429 | #13 | 0.034 | Not Required | Pass |
| 3 | 0.008 | 0.504 | 0.047 | 0.049 | 0.006 | 0.514 | #13 | 0.044 | Not Required | Pass |
| 4 | 0.007 | 0.503 | 0.167 | 0.050 | 0.020 | 0.614 | #13 | 0.078 | Not Required | Pass |

| | | | | | | | | | | |
|-----|-------|-------|-------|-------|-------|-------|-----|--------------|--------------|------|
| 4 | 0.007 | 0.501 | 0.107 | 0.050 | 0.050 | 0.014 | #13 | 0.070 | Not Required | Pass |
| 5 | 0.008 | 0.314 | 0.139 | 0.050 | 0.028 | 0.333 | #13 | 0.073 | Not Required | Pass |
| 6 | 0.013 | 0.690 | 0.119 | 0.071 | 0.019 | 0.780 | #13 | 0.044 | Not Required | Pass |
| 7 | 0.013 | 0.428 | 0.261 | 0.069 | 0.052 | 0.476 | #13 | 0.073 | Not Required | Pass |
| 8 | 0.003 | 0.098 | 0.154 | 0.046 | 0.013 | 0.160 | #21 | 0.088 | Not Required | Pass |
| 9 | 0.013 | 0.053 | 0.089 | 0.003 | 0.004 | 0.139 | #13 | 0.198 | Not Required | Pass |
| 10 | 0.014 | 0.688 | 0.247 | 0.069 | 0.042 | 0.746 | #13 | 0.078 | Not Required | Pass |
| 11 | 0.004 | 0.097 | 0.160 | 0.046 | 0.013 | 0.169 | #21 | 0.088 | Not Required | Pass |
| 12 | 0.003 | 0.403 | 0.290 | 0.087 | 0.055 | 0.694 | #13 | 0.052 | Not Required | Pass |
| 13 | 0.005 | 0.168 | 0.342 | 0.056 | 0.016 | 0.459 | #21 | 0.265 | Not Required | Pass |
| 14 | 0.006 | 0.170 | 0.336 | 0.056 | 0.016 | 0.449 | #21 | 0.177 | Not Required | Pass |
| 15 | 0.000 | 0.055 | 0.087 | 0.023 | 0.007 | 0.129 | #21 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.055 | 0.087 | 0.023 | 0.007 | 0.129 | #21 | Not Required | Not Required | Pass |
| 101 | 0.046 | 0.678 | 0.048 | 0.036 | 0.004 | 0.721 | #13 | 0.584 | Not Required | Pass |
| 102 | 0.003 | 0.403 | 0.290 | 0.087 | 0.055 | 0.694 | #13 | 0.052 | Not Required | Pass |
| 103 | 0.013 | 0.690 | 0.119 | 0.071 | 0.019 | 0.780 | #13 | 0.044 | Not Required | Pass |
| 104 | 0.014 | 0.688 | 0.247 | 0.069 | 0.042 | 0.746 | #13 | 0.078 | Not Required | Pass |
| 105 | 0.013 | 0.428 | 0.261 | 0.069 | 0.052 | 0.476 | #13 | 0.073 | Not Required | Pass |
| 106 | 0.008 | 0.504 | 0.047 | 0.049 | 0.006 | 0.514 | #13 | 0.044 | Not Required | Pass |
| 107 | 0.008 | 0.314 | 0.139 | 0.050 | 0.028 | 0.333 | #13 | 0.073 | Not Required | Pass |
| 108 | 0.000 | 0.055 | 0.087 | 0.023 | 0.007 | 0.129 | #21 | Not Required | Not Required | Pass |
| 109 | 0.013 | 0.053 | 0.089 | 0.003 | 0.004 | 0.139 | #13 | 0.198 | Not Required | Pass |
| 110 | 0.007 | 0.501 | 0.167 | 0.051 | 0.030 | 0.614 | #13 | 0.078 | Not Required | Pass |
| 111 | 0.000 | 0.055 | 0.087 | 0.023 | 0.007 | 0.129 | #21 | Not Required | Not Required | Pass |
| 112 | 0.004 | 0.218 | 0.209 | 0.055 | 0.043 | 0.429 | #13 | 0.034 | Not Required | Pass |
| 113 | 0.005 | 0.168 | 0.342 | 0.056 | 0.016 | 0.459 | #21 | 0.177 | Not Required | Pass |
| 114 | 0.006 | 0.170 | 0.336 | 0.056 | 0.016 | 0.449 | #21 | 0.265 | Not Required | Pass |
| 115 | 0.009 | 0.451 | 0.192 | 0.046 | 0.013 | 0.576 | #13 | 0.557 | Not Required | Pass |
| 116 | 0.004 | 0.455 | 0.192 | 0.046 | 0.013 | 0.581 | #13 | 0.557 | Not Required | Pass |

Definitions

| | |
|----------|---|
| Φ_t | Safety factor for tensile |
| Φ_c | Safety factor for compression |
| Φ_b | Safety factor for flexure |
| Φ_v | Safety factor for shear |
| E | Modulus of elasticity |
| F_y | Specified minimum yield stress |
| F_u | Specified minimum tensile strength |
| A | Cross-sectional area |
| J | Torsional constant |
| I_{yp} | Moment of inertia about the Y axes |
| I_{zp} | Moment of inertia about the Z axes |
| I_w | Warping constant |
| S_{yp} | Plastic section modulus about the Y axis |
| S_{zp} | Plastic section modulus about the Z axis |
| KL | Effective length |
| C_b | Buckling modification factor (from all load combinations) |
| L_b | Length between braced points |
| LST | Limited slenderness for tension |
| LSC | Limited slenderness for compression |
| LD | Limited deflection |
| P_n | Nominal axial strength (tension/compression) |
| M_n | Nominal flexural strength (about Z/Y axis) |
| V_n | Nominal shear strength (along Z/Y axis) |
| P | Design ratio in case of axial force |
| M_z | Design ratio in case of bending about Z axis |
| M_y | Design ratio in case of bending about Y axis |
| V_y | Design ratio in case of shear along Y axis |

| | |
|-----------------|--|
| V_z | Design ratio in case of shear along Z axis |
| (P, M_z, M_y) | Design ratio in case of axial force and bending action |
| KL/r | Design ratio in case of section slenderness |
| δ | Design ratio in case of member deflection |
| OK | Capacity is provided |
| NG | Capacity is not provided |

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

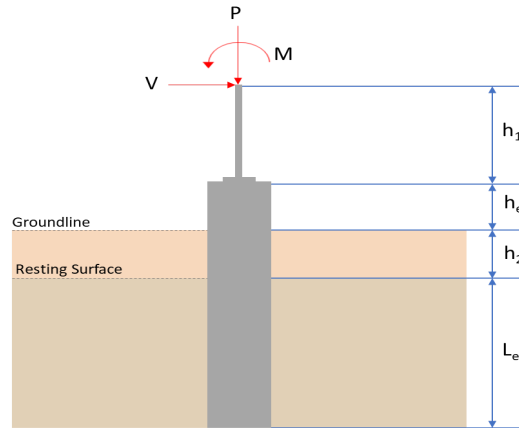
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 4.492 | 6.445 |
| V_x (kip) | -2.418 | -4.030 |
| V_z (kip) | 0.257 | 0.415 |
| M_x (kipft) | 0.762 | 1.220 |
| M_z (kipft) | 24.971 | 42.220 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.418 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.38503 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.971 \text{ kipft}) + ((-2.418 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.9763 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.7102 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.040924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.762 \text{ kipft}) + ((0.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12134 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 2.5134 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.7102 \text{ ft}), (2.5134 \text{ ft})]$$

$$L_{e,req} = 5.71 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.71 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9136$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.492 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28075 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28075 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14037$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.38503 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.9763 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9763 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.9763 \text{ kipft/ft})) + (4 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3164 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9763 \text{ kipft/ft})) + (3 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.9763 \text{ kipft/ft})) + (2 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20356 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.9763 \text{ kipft/ft})) + ((-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85188 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3164 \text{ ft})}{2}$$

$$p_a = 0.32373 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20356 \text{ kip/ft}^2)}{(0.32373 \text{ kip/ft}^2)}$$

$$Ratio = 0.62878$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.85188 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.90867$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = 0.040924 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.12134 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.040924 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.12134 \text{ kipft/ft})) + (4 \times (0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.471 \text{ ft}$$

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

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

$$p = \frac{0.75 [(4 \times (0.12134 \text{ kipft/ft})) + (3 \times (0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.12134 \text{ kipft/ft})) + (2 \times (0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.03441 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.12134 \text{ kipft/ft})) + ((0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.076562 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.471 \text{ ft})}{2}$$

$$p_a = 0.33532 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.03441 \text{ kip/ft}^2)}{(0.33532 \text{ kip/ft}^2)}$$

$$Ratio = 0.10262$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

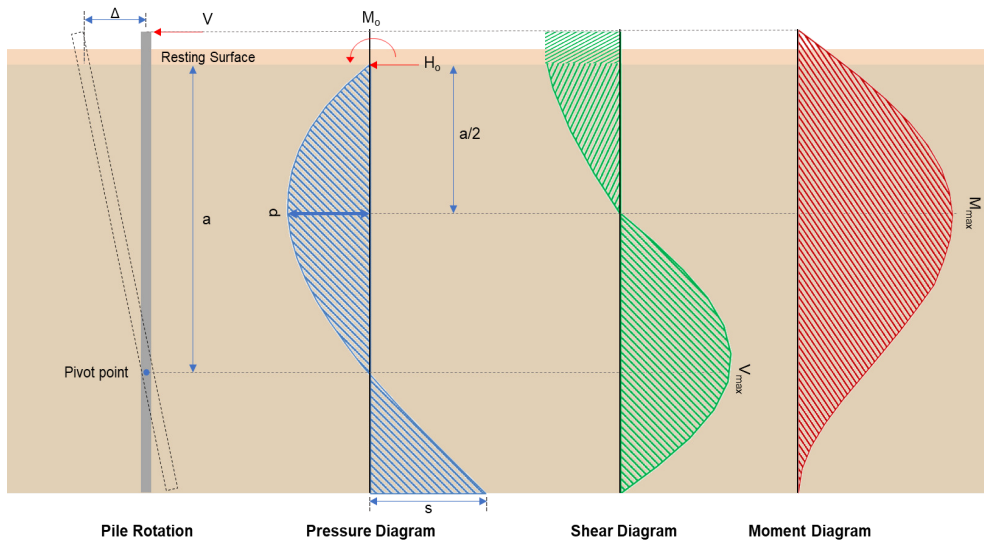
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.076562 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.081666$$

Status: **PASS**
Ratio: **0.100**

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.64172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.22 \text{ kipft}) + ((-4.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7229 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7229 \text{ kipft/ft})}{(-0.64172 \text{ kip/ft})}$$

$$E = 10.476 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7229 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.7229 \text{ kipft/ft})) + (4 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.64172 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.7229 \text{ kip/ft})) + (4 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3149 \text{ ft}$$

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

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

$$V_{max} = ((-0.64172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.3064 \text{ kip}$$

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

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

$$M_{max} = ((-0.64172 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(10.476 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.476 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.627 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.415 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.066083 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.22 \text{ kipft}) + ((0.415 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19427 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19427 \text{ kipft/ft})}{(0.066083 \text{ kip/ft})}$$

$$E = 2.9398 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19427 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.066083 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19427 \text{ kipft/ft})) + (4 \times (0.066083 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.472 \text{ ft}$$

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

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

$$V_{max} = ((0.066083 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.472 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.472 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39628 \text{ kip}$$

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

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

$$M_{max} = ((0.066083 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.9398 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0912 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.445 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.382 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.382 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.445 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0024092$</p> | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.445 \text{ kip} \rightarrow 6445 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6445 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.34 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.34 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.34 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.34 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.66 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.3064 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(9.3064 \text{ kip})}{(110.66 \text{ kip})}$$

$$Ratio = 0.084103$$

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

Considering z-direction:

$V_{max} = 0.39628 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.39628 \text{ kip})}{(110.66 \text{ kip})}$$

$$Ratio = 0.0035813$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 27.627 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(27.627 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11068$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$M_{max} = 1.0912 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.0912 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0043716$$

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

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

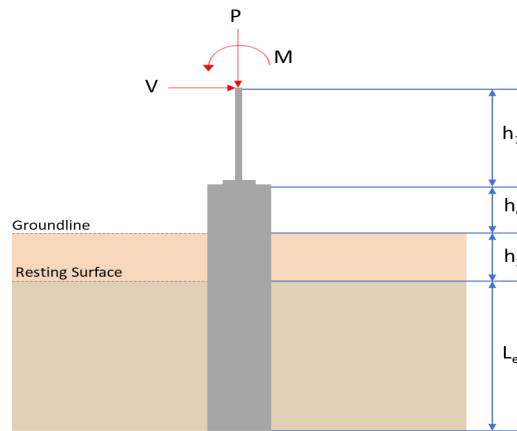
SkyCiv Foundation Design

Pile Foundation

Design Information :

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

Pile Input



Geometry

Pile shape: rectangular

$b = 48$ in - Pile width

$D = 48$ in - Pile depth

$L = 6.25$ ft - Total pile length

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

$h_2 = 0$ ft - Depth to resisting surface

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

Tabulation of Soil Parameters

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

Tabulation of Loads

| Load Component | ASD | LRFD |
|----------------|--------|--------|
| P (kip) | 4.492 | 6.445 |
| V_x (kip) | -2.418 | -4.030 |
| V_z (kip) | -0.257 | -0.415 |
| M_x (kipft) | -0.762 | -1.220 |
| M_z (kipft) | 24.971 | 42.221 |

Material Properties

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

Required depth to resist lateral loads (ASD)

H - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

Considering x-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.418 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.38503 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(24.971 \text{ kipft}) + ((-2.418 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.9763 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 5.7102 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.257 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.040924 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.762 \text{ kipft}) + ((-0.257 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.12134 \text{ kipft/ft}$$

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:

$L_{e,z} = 1.7544 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.7102 \text{ ft}), (1.7544 \text{ ft})]$$

$$L_{e,req} = 5.71 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.71 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.9136$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(4.492 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.28075 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.28075 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.14037$$

Status: **PASS**
Ratio: **0.140**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.38503 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.9763 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.9763 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.9763 \text{ kipft/ft})) + (4 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3164 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.9763 \text{ kipft/ft})) + (3 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.9763 \text{ kipft/ft})) + (2 \times (-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.20356 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.9763 \text{ kipft/ft})) + ((-0.38503 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.85188 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.3164 \text{ ft})}{2}$$

$$p_a = 0.32373 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.20356 \text{ kip/ft}^2)}{(0.32373 \text{ kip/ft}^2)}$$

$$Ratio = 0.62878$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(0.85188 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.90867$$

Status: **PASS**
Ratio: **0.630**

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

Considering z-direction:

$H_o = -0.040924 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.12134 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.12134 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.040924 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.12134 \text{ kipft/ft})) + (4 \times (-0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.471 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.12134 \text{ kipft/ft})) + (3 \times (-0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (0.12134 \text{ kipft/ft})) + (2 \times (-0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = -0.010347 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.12134 \text{ kipft/ft})) + ((-0.040924 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = -0.0020117 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.471 \text{ ft})}{2}$$

$$p_a = 0.33532 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.010347 \text{ kip/ft}^2)}{(0.33532 \text{ kip/ft}^2)}$$

$$Ratio = -0.030857$$

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

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

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

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

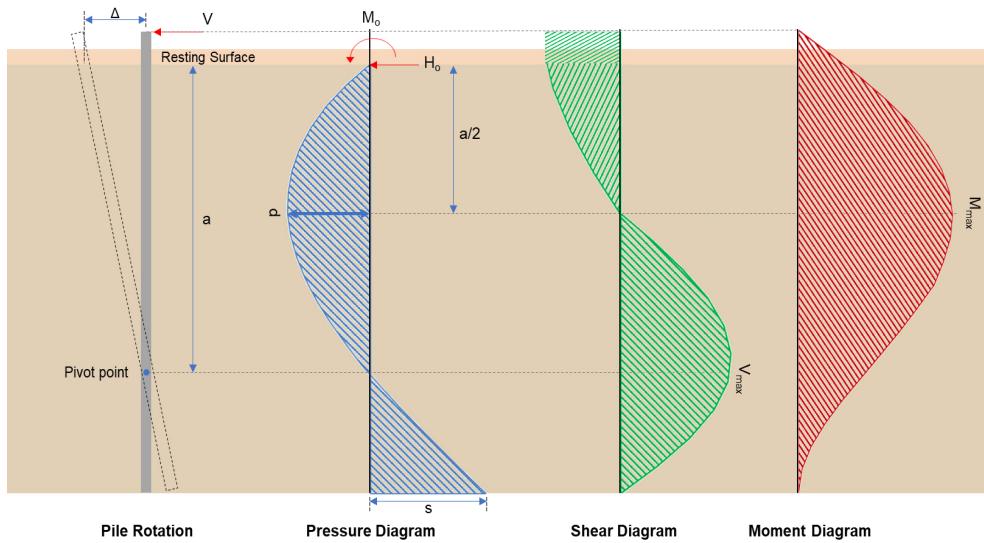
Ratio - Lateral soil capacity

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

$$Ratio = \frac{(-0.0020117 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = -0.0021458$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-4.03 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.64172 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(42.221 \text{ kipft}) + ((-4.03 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.7231 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.7231 \text{ kipft/ft})}{(-0.64172 \text{ kip/ft})}$$

$$E = 10.477 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.7231 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.7231 \text{ kipft/ft})) + (4 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = \frac{(-0.64172 \text{ kip/ft}) \times (48 \text{ in})}{(6 \times (6.7231 \text{ kip/ft})) + (4 \times (-0.64172 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.3149 \text{ ft}$$

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

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

$$V_{max} = ((-0.64172 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.477 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (10.477 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 9.3066 \text{ kip}$$

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

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

$$M_{max} = ((-0.64172 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(10.477 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (10.477 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (10.477 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.3149 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 27.627 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.415 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.066083 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.22 \text{ kipft}) + ((-0.415 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19427 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19427 \text{ kipft/ft})}{(-0.066083 \text{ kip/ft})}$$

$$E = 2.9398 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19427 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.066083 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19427 \text{ kipft/ft})) + (4 \times (-0.066083 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.472 \text{ ft}$$

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

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

$$V_{max} = ((-0.066083 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.472 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.472 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.39628 \text{ kip}$$

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

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

$$M_{max} = ((-0.066083 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(2.9398 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (2.9398 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.472 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 1.0912 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

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

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

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

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

Longitudinal reinforcement:

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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.445 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.382 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.382 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|---|---|--|
| <p>25.2.3</p> <p>25.7.2.2</p> <p>25.7.2.1</p> | <p style="text-align: center;">$Ratio = 0.96556$</p> <p>$s_{rebar} = Max[1.5, (1.5 d_{bar})]$</p> <p>$s_{rebar} = Max[1.5, (1.5 \times (0.625 \text{ in}))]$</p> <p>$s_{rebar} = 1.5 \text{ in}$</p> <p>Ties:</p> <p>Since longitudinal reinforcement is \leq No. 10ø: Use #3(0.375 in)</p> <p>$s_{ties} = Min[(16 d_{bar}), (48 d_{ties}), Min(D, b)]$</p> <p>$s_{ties} = Min[(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min((48 \text{ in}), (48 \text{ in}))]$</p> <p>$s_{ties} = 10 \text{ in}$</p> <p>Summary:</p> <p style="text-align: center;">Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p>ϕP_N - Allowable axial compressive strength</p> <p style="text-align: center;">$\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_y k A_{st})]$</p> <p style="text-align: center;">$\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (2.5 \text{ ksi}) \times [(2304 \text{ in}^2) - (4.2951 \text{ in}^2)]) + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$</p> <p style="text-align: center;">$\phi P_N = 2675.2 \text{ kip}$</p> <p>Ratio - Capacity</p> <p style="text-align: center;">$Ratio = \frac{P}{\phi P_N}$</p> <p style="text-align: center;">$Ratio = \frac{(6.445 \text{ kip})}{(2675.2 \text{ kip})}$</p> <p style="text-align: center;">$Ratio = 0.0024092$</p> | <p>Status: PASS Ratio: 0.000</p> |
| <p>22.5.2.2</p> <p>22.5.5.1.3</p> <p>22.5.5.1.1</p> | <p>Shear Strength (ACI 318-19, LRFD)</p> <p>Parameters:</p> <p>$b_w = 48 \text{ in}$ - Effective width, d - Effective depth</p> <p style="text-align: center;">$d = 0.80 D$</p> <p style="text-align: center;">$d = 0.80 \times (48 \text{ in})$</p> <p style="text-align: center;">$d = 38.4 \text{ in}$</p> <p>λ_s - size effect modification factor</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{d}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = MIN \left[\sqrt{\frac{2}{1 + \frac{(38.4 \text{ in})}{10}}}, 1 \right]$</p> <p style="text-align: center;">$\lambda_s = 0.64282$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$,</p> <p>$V_{c,max}$ - Max shear strength of concrete</p> <p style="text-align: center;">$V_{c,max} = 5 \lambda_s \sqrt{f'_{ck}} b_w d$</p> <p style="text-align: center;">$V_{c,max} = 5 \times (0.64282) \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$</p> | |

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 6.445 \text{ kip} \rightarrow 6445 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi}) + \frac{(6445 \text{ lbf})}{6 \times (2304 \text{ in}^2)}} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.34 \text{ kip}$$

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

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.34 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.34 \text{ kip}$$

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

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

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

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

A_v - Ties rebar area,

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

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

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

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

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

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

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

V_s - Governing shear strength of steel

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

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

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

22.5.1.1 ϕV_n - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.34 \text{ kip}) + (50.894 \text{ kip}))$$

$$\phi V_n = 110.66 \text{ kip}$$

Considering x-direction:

$V_{max} = 9.3066 \text{ kip}$ - Maximum shear force in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(9.3066 \text{ kip})}{(110.66 \text{ kip})}$$

$$Ratio = 0.084105$$

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

Considering z-direction:

$V_{max} = 0.39628 \text{ kip}$ - Maximum shear force in the z-direction,
Ratio - Capacity

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

$$Ratio = \frac{(0.39628 \text{ kip})}{(110.66 \text{ kip})}$$

$$Ratio = 0.0035813$$

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

Flexural Strength (ACI 318-19, LRFD)

S_m - Section modulus

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

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

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

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

Allowable flexural strength:

M_n shall be the lesser of:

$\phi M_{n,1}$

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

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

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

14.5.2.1b

$\phi M_{n,2}$

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

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

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

Therefore,

ϕM_n - Allowable flexural strength,

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

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

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

Considering x-direction:

$M_{max} = 27.627 \text{ kipft}$ - Maximum moment in the x-direction,

Ratio - Capacity

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

$$Ratio = \frac{(27.627 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.11069$$

Status: **PASS**
Ratio: **0.110**

Considering z-direction:

$M_{max} = 1.0912 \text{ kipft}$ - Maximum moment in the z-direction,

Ratio - Capacity

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

$$Ratio = \frac{(1.0912 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.0043716$$

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