

Your Project Calculations



Project Name: USDCampusPhase1-E2

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=USDCampusPhase1-E2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/6_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=oPckOw3LX8vQc2ZbW51jw68JvgHqctTV25CtQn6cnTBUY8rKXhR4c8727dPiK5D

Array Specification

| | |
|------------------------------------|----------------------------------|
| Product: | Beam |
| Unique ID: | 3P-19.75-8TOP-SD-45-L-5Hx8W-5EHD |
| Duty Classification: | SD |
| Module Width: | 44.60 in |
| Module Length: | 82.60in |
| Number of Rows: | 5 |
| Number of Columns: | 8 |
| Total Number of Modules: | 40 |
| Desired Tilt Angle: | 10 |
| Front Edge Clearance: | 15 |
| Total Array Height at Tilt: | 18.25 ft |
| Total Frame Length: | 54.50 ft |
| Frame Weight: | 2865 lbs |
| Array Dimensions N/S: | 18.79 ft |
| Array Dimensions E/W: | 55.73 ft |
| Rail Length: | 225.50 in |
| Rail Spacing: | 3.48 ft |
| Rail Check: | PASS (46% utilized) |

Support Specifications

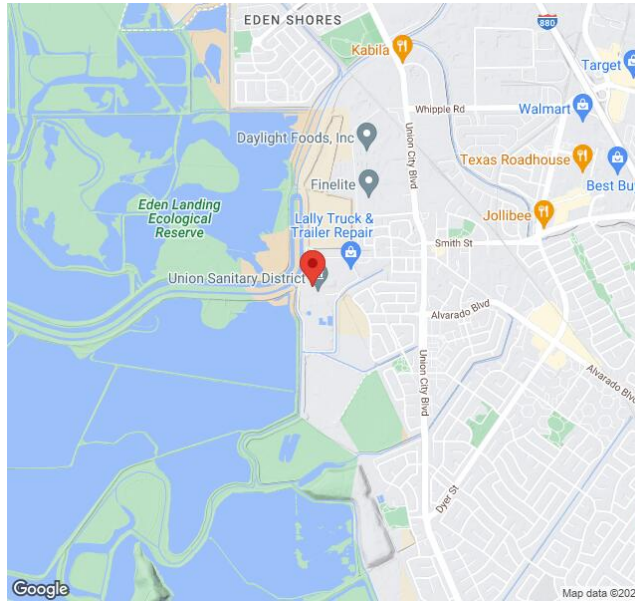
| | |
|---------------------------------|-----------------|
| Pole Size: | 8in Pipe Sch 40 |
| Pole Length above Grade: | 16.63 ft |
| Number of Poles: | 3 |
| Pole Spacing: | 19.75 ft |

Foundation Specifications

| | |
|--|---|
| Foundation Type: | Round |
| Foundation Dimensions: | Ø36 in |
| Foundation Depth (below grade): | Pile 1: 6.75 ft Pile 2: 7.00 ft Pile 3: 6.75 ft |
| Foundation Volume: | 5.367 y ³ |
| Foundation Result: | PASSED |
| Mount Twist: | 0.071350 kip |

Site Info

| | |
|-----------------------------------|---|
| Risk Category: | I |
| Exposure: | B |
| Soil Classification: | sand |
| Site Location: | 5072 Benson Rd, Union City, CA 94587, USA |
| Wind Speed: | 85 mph |
| Snow Load: | 0 psf |
| Design Uplift Pressure: | Multiple pressures |
| Design Downforce Pressure: | Multiple pressures |
| Design Snow Pressure: | 0.000000 ksf |



Design Disclaimer

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

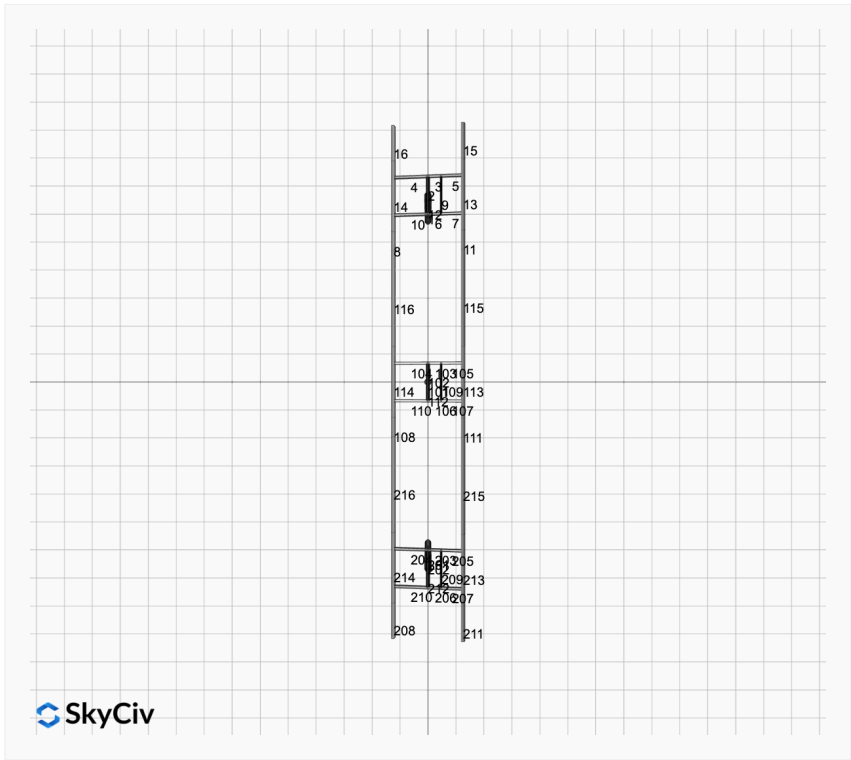
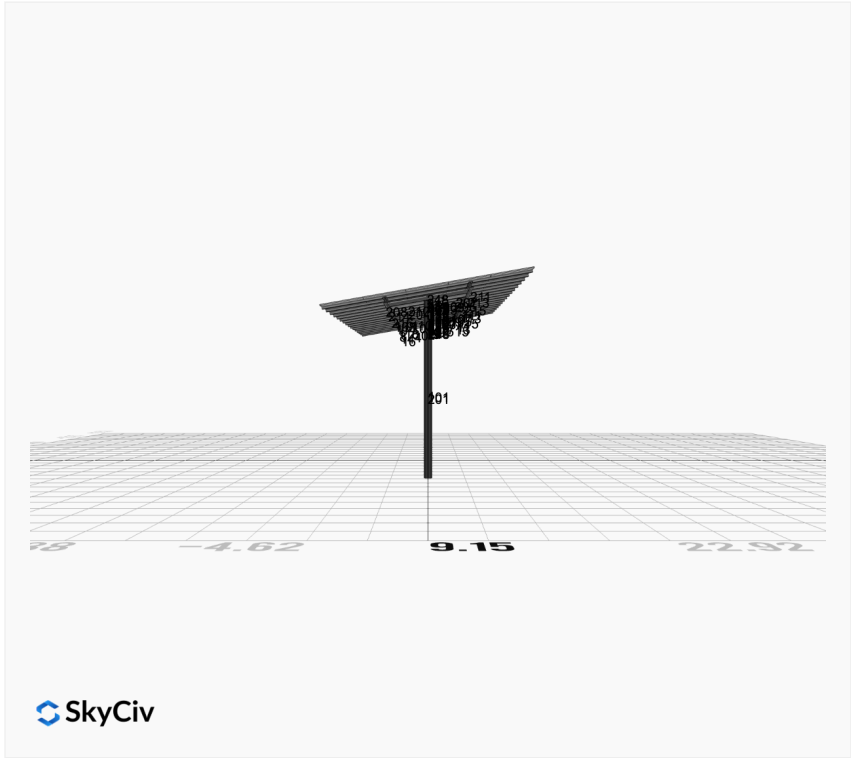
AutoDesigner Input

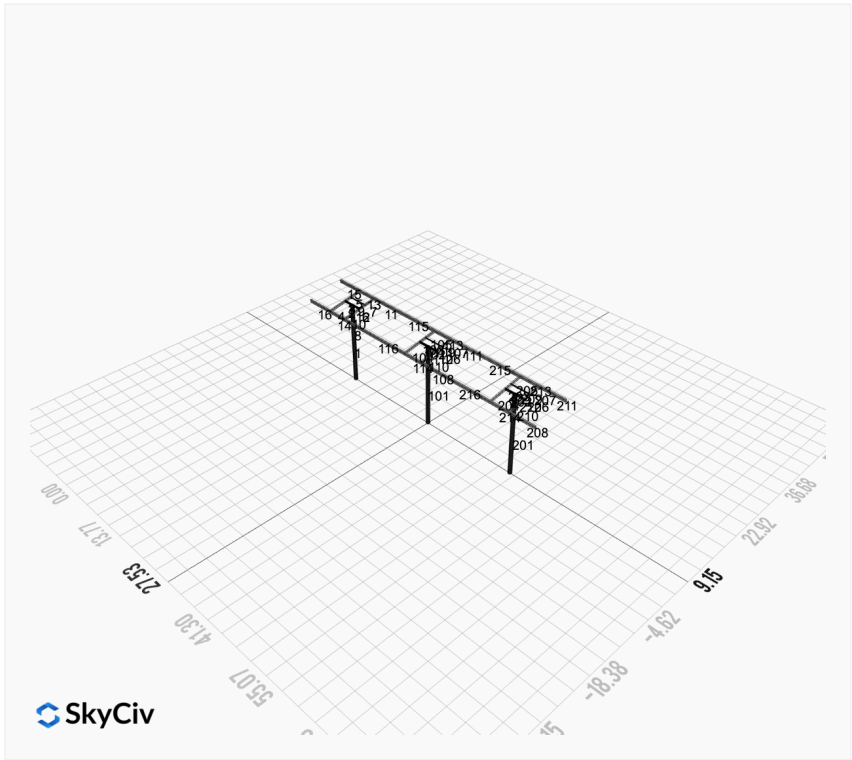
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Design Notes:

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







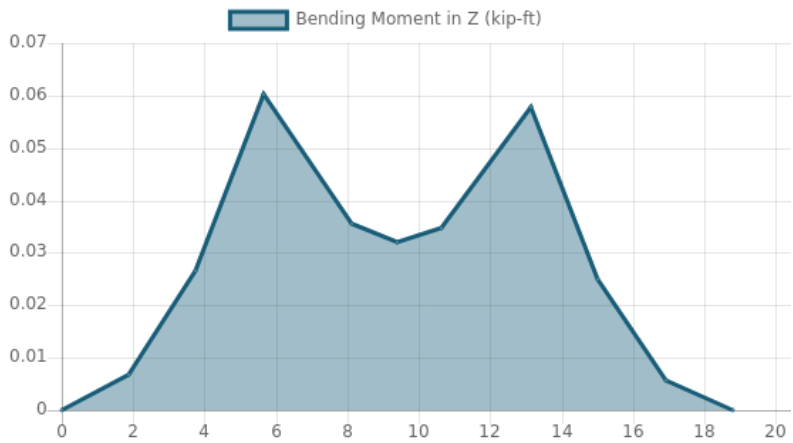
Rail Design Check

Rail Length: 18.79166666666668 ft
Additional Restraints Required: None
Tributary Width: 3.483333333333333 ft
Material: Aluminium
Density: 169 lb/ft³
Elasticity Modulus: 10000 ksi
Fy: 34.5 ksi
Fu: 37 ksi
Wind uplift Case A (X): 0.0000 kip/ft
Wind uplift Case A (Y): 0.0303 kip/ft
Wind uplift Case A: 0.0193 kip/ft
Wind uplift Case B: 0.0193 kip/ft
Wind uplift Case B (X): 0.0000 kip/ft
Wind uplift Case B (Y): 0.0432 kip/ft

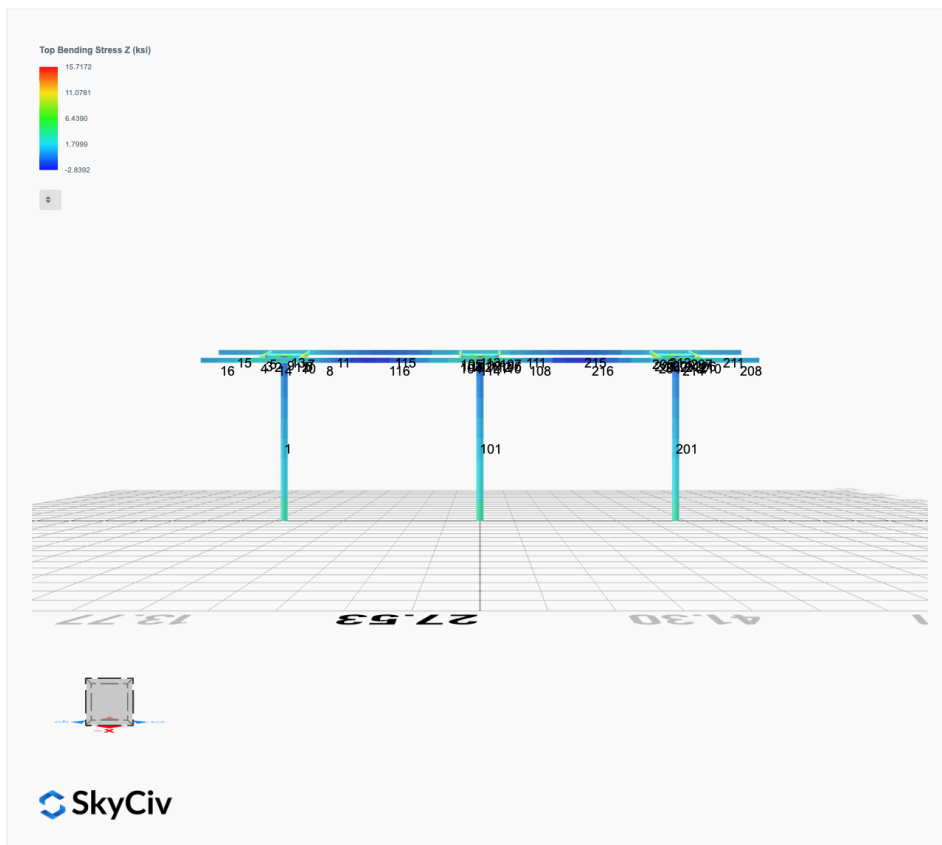
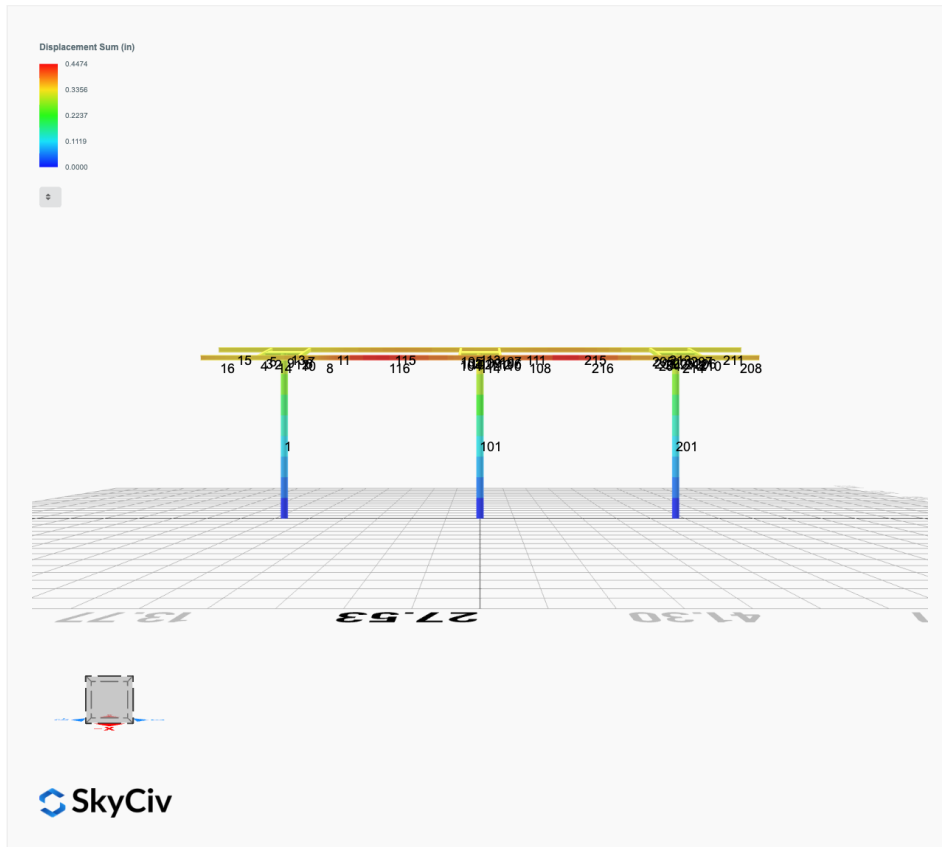


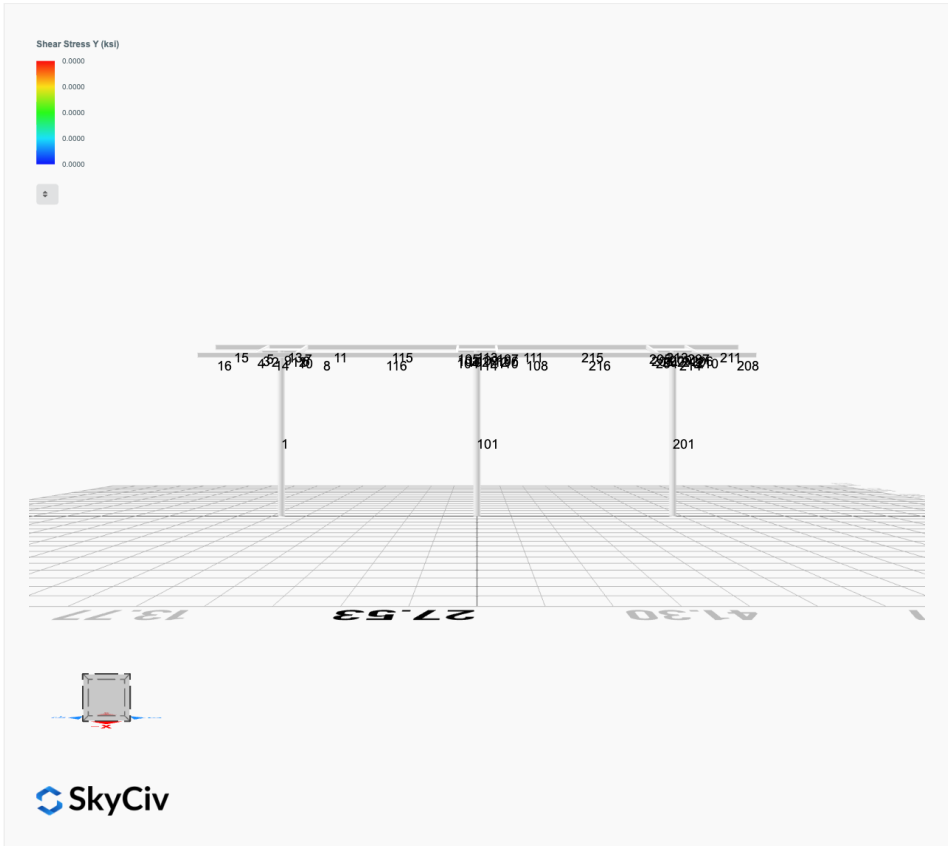
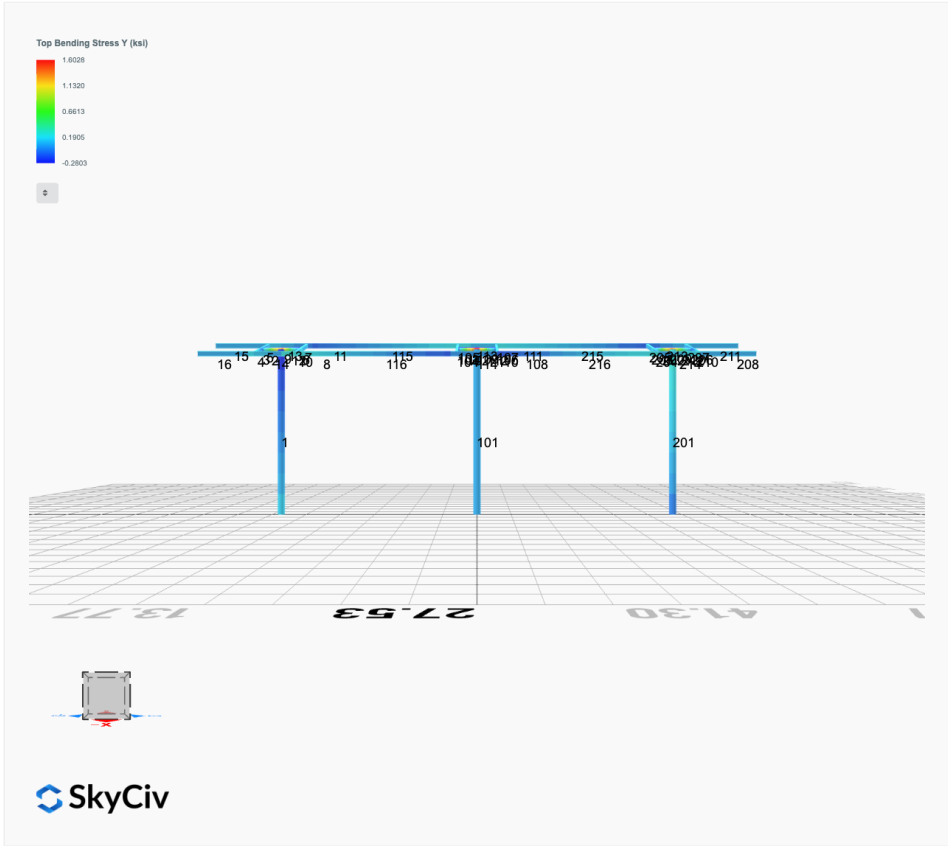
| Result Check | Max Limit | Max Value | Utility | Status |
|---------------------|-----------|-------------|---------|--------|
| Custom Stress Limit | 34.5 | 15.71619624 | 0.456 | PASS |
| Material Yield | 34.5 | 15.71619624 | 0.456 | PASS |
| Material Strength | 37 | 15.71619624 | 0.425 | PASS |

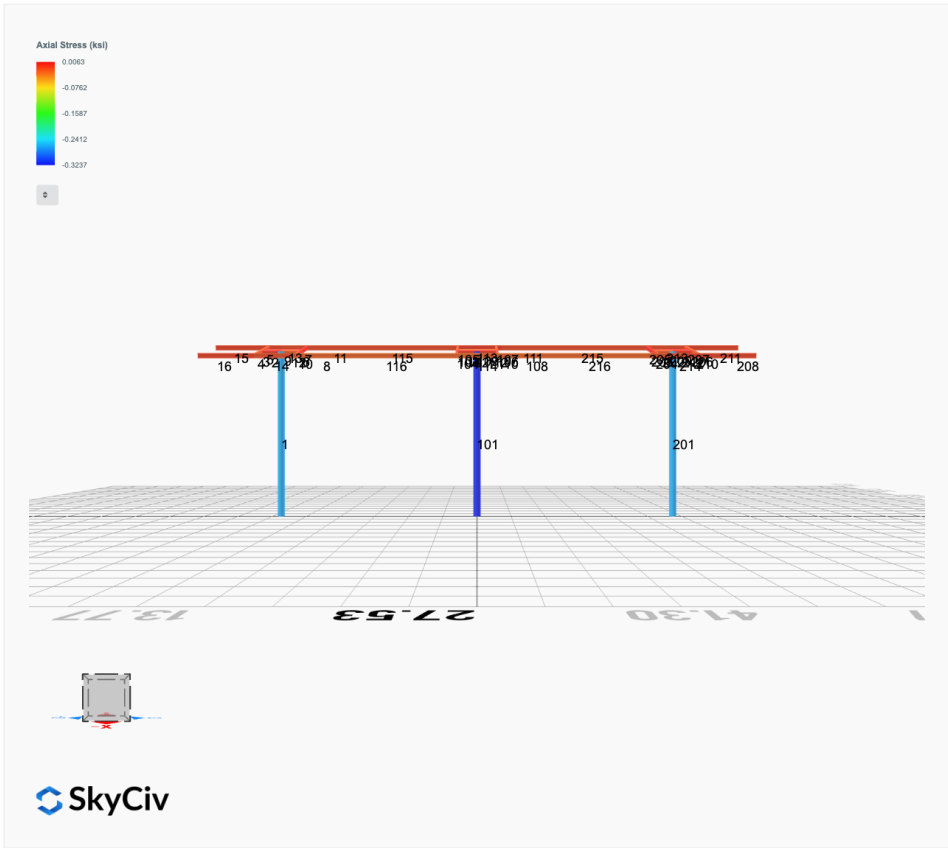
Member 1, ULS: 1. 1.4D



FEM Results (Envelope Worst Case for each member)







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

ASD Load Combination Results

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|--------|---------|---------|---------|---------|
| ULS: 1. D | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 2. D + L | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 5b. D + 0.7E | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0024 | 2.4066 | 0.0251 | 0.1326 | -0.0062 | -0.0148 |
| ULS: 8. 0.6D + 0.7E | 0.0015 | 1.4439 | 0.0151 | 0.0796 | -0.0037 | -0.0089 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.3570 | 4.3849 | 0.0556 | 0.2917 | -0.0441 | 7.6465 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.3570 | 4.3849 | 0.0556 | 0.2917 | -0.0441 | 7.6465 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.2502 | 1.0202 | 0.0050 | 0.0276 | 0.0158 | -2.9263 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.2276 | 1.1975 | 0.0050 | 0.0282 | 0.0232 | -8.2959 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.2671 | 3.8903 | 0.0480 | 0.2519 | -0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2671 | 3.8903 | 0.0480 | 0.2519 | -0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.1883 | 1.3668 | 0.0100 | 0.0539 | 0.0103 | -2.1984 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1713 | 1.4998 | 0.0100 | 0.0543 | 0.0159 | -6.2256 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.2671 | 3.8903 | 0.0480 | 0.2519 | -0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2671 | 3.8903 | 0.0480 | 0.2519 | -0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.1883 | 1.3668 | 0.0100 | 0.0539 | 0.0103 | -2.1984 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1713 | 1.4998 | 0.0100 | 0.0543 | 0.0159 | -6.2256 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.3579 | 3.4223 | 0.0456 | 0.2387 | -0.0416 | 7.6524 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.3579 | 3.4223 | 0.0456 | 0.2387 | -0.0416 | 7.6524 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.2493 | 0.0576 | -0.0051 | -0.0254 | 0.0183 | -2.9204 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.2266 | 0.2349 | -0.0051 | -0.0249 | 0.0257 | -8.2899 |

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 | 6.1852 |
| Shear X | -0.5990 |
| Shear Z | 0.0813 |
| Moment X | 0.4264 |
| Moment Y (Twist) | 0.0713 |
| Moment Z | 14.1583 |

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 | 4.3849 |
| Shear X | -0.3579 |
| Shear Z | 0.0556 |
| Moment X | 0.2917 |
| Moment Y (Twist) | 0.0441 |
| Moment Z | 8.2959 |

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 | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 2. D + L | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 3. D + (S or Lr or R) | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 3. D + (S or Lr or R) | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 5b. D + 0.7E | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|--------|---------|--------|--------|---------|
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0049 | 2.7207 | -0.0000 | 0.0000 | 0.0000 | 0.0963 |
| ULS: 8. 0.6D + 0.7E | -0.0029 | 1.6324 | -0.0000 | 0.0000 | 0.0000 | 0.0578 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.3934 | 5.0439 | -0.0000 | 0.0000 | 0.0000 | 8.4253 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.3934 | 5.0439 | -0.0000 | 0.0000 | 0.0000 | 8.4253 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.2761 | 1.0893 | -0.0000 | 0.0000 | 0.0000 | -3.1577 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.2207 | 1.3057 | -0.0000 | 0.0000 | 0.0000 | -8.7392 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.2963 | 4.4631 | -0.0000 | 0.0000 | 0.0000 | 6.3430 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2963 | 4.4631 | -0.0000 | 0.0000 | 0.0000 | 6.3430 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2058 | 1.4972 | -0.0000 | 0.0000 | 0.0000 | -2.3442 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1643 | 1.6594 | -0.0000 | 0.0000 | 0.0000 | -6.5303 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.2963 | 4.4631 | -0.0000 | 0.0000 | 0.0000 | 6.3430 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2963 | 4.4631 | -0.0000 | 0.0000 | 0.0000 | 6.3430 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.2058 | 1.4972 | -0.0000 | 0.0000 | 0.0000 | -2.3442 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1643 | 1.6594 | -0.0000 | 0.0000 | 0.0000 | -6.5303 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.3915 | 3.9556 | -0.0000 | 0.0000 | 0.0000 | 8.3868 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.3915 | 3.9556 | -0.0000 | 0.0000 | 0.0000 | 8.3868 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.2780 | 0.0010 | -0.0000 | 0.0000 | 0.0000 | -3.1962 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.2226 | 0.2174 | -0.0000 | 0.0000 | 0.0000 | -8.7777 |

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 | 7.1366 |
| Shear X | -0.6521 |
| Shear Z | -0.0000 |
| Moment X | 0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 14.9816 |

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 | 5.0439 |
| Shear X | -0.3934 |
| Shear Z | -0.0000 |
| Moment X | 0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 8.7777 |

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 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 2. D + L | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 3. D + (S or Lr or R) | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 5b. D + 0.7E | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0024 | 2.4066 | -0.0251 | -0.1326 | 0.0062 | -0.0148 |
| ULS: 8. 0.6D + 0.7E | 0.0015 | 1.4439 | -0.0151 | -0.0796 | 0.0037 | -0.0089 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -0.3570 | 4.3849 | -0.0556 | -0.2917 | 0.0441 | 7.6465 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -0.3570 | 4.3849 | -0.0556 | -0.2917 | 0.0441 | 7.6465 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 0.2502 | 1.0202 | -0.0050 | -0.0276 | -0.0158 | -2.9263 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.2276 | 1.1975 | -0.0050 | -0.0282 | -0.0232 | -8.2959 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -0.2671 | 3.8903 | -0.0480 | -0.2519 | 0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2671 | 3.8903 | -0.0480 | -0.2519 | 0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.1883 | 1.3668 | -0.0100 | -0.0539 | -0.0103 | -2.1984 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1713 | 1.4998 | -0.0100 | -0.0543 | -0.0159 | -6.2256 |

| 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 | -0.2671 | 3.8903 | -0.0480 | -0.2519 | 0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -0.2671 | 3.8903 | -0.0480 | -0.2519 | 0.0346 | 5.7312 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 0.1883 | 1.3668 | -0.0100 | -0.0539 | -0.0103 | -2.1984 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.1713 | 1.4998 | -0.0100 | -0.0543 | -0.0159 | -6.2256 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -0.3579 | 3.4223 | -0.0456 | -0.2387 | 0.0416 | 7.6524 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -0.3579 | 3.4223 | -0.0456 | -0.2387 | 0.0416 | 7.6524 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 0.2493 | 0.0576 | 0.0051 | 0.0254 | -0.0183 | -2.9204 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.2266 | 0.2349 | 0.0051 | 0.0249 | -0.0257 | -8.2899 |

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 | 6.1852 |
| Shear X | -0.5990 |
| Shear Z | -0.0813 |
| Moment X | -0.4264 |
| Moment Y (Twist) | 0.0713 |
| Moment Z | 14.1584 |

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 | 4.3849 |
| Shear X | -0.3579 |
| Shear Z | -0.0556 |
| Moment X | -0.2917 |
| Moment Y (Twist) | 0.0441 |
| Moment Z | 8.2959 |

Project Details

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



Design Input Information

| Design Factors | | | |
|----------------|----------|----------|----------|
| 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 | | | | |
| 9 | 8in Pipe Sch 40 | 8.63 | 0.32 | | | | |



| 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_{yp} (in ⁴) | I_{zp} (in ⁴) | I_w (in ⁶) | S_{yp} (in ³) | S_{zp} (in ³) |
|----|-----------------|----------------------|----------------------|-----------------------------|-----------------------------|--------------------------|-----------------------------|-----------------------------|
| 1 | 2in Pipe Sch 40 | 1.07 | 1.33 | 0.67 | 0.67 | 0.00 | 0.76 | 0.76 |
| 4 | 4in Pipe Sch 40 | 3.17 | 14.47 | 7.23 | 7.23 | 0.00 | 4.31 | 4.31 |
| 9 | 8in Pipe Sch 40 | 8.40 | 144.98 | 72.49 | 72.49 | 0.00 | 22.21 | 22.21 |

Member Design Capacity

| Member ID | $\Phi_t P_n$ (kip) | $\Phi_c P_n$ (kip) | $\Phi_b M_{zn}$ (k-ft) | $\Phi_b M_{yn}$ (k-ft) | $\Phi_v V_{yn}$ (kip) | $\Phi_v V_{zn}$ (kip) |
|-----------|--------------------|--------------------|------------------------|------------------------|-----------------------|-----------------------|
| 1 | 377.97 | 93.23 | 83.29 | 83.29 | 113.39 | 113.39 |
| 2 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 3 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 4 | 79.65 | 72.01 | 10.99 | 4.60 | 29.14 | 16.61 |
| 5 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 6 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 7 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 8 | 120.60 | 117.88 | 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 | 4.60 | 29.14 | 16.61 |
| 11 | 120.60 | 117.88 | 23.36 | 6.45 | 30.09 | 45.74 |
| 12 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 13 | 120.60 | 98.23 | 18.31 | 6.45 | 30.09 | 45.74 |
| 14 | 120.60 | 98.23 | 19.19 | 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 | 377.97 | 93.23 | 83.29 | 83.29 | 113.39 | 113.39 |
| 102 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 103 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 104 | 79.65 | 72.01 | 10.99 | 4.60 | 29.14 | 16.61 |
| 105 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 106 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 107 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 108 | 120.60 | 117.88 | 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 | 4.60 | 29.14 | 16.61 |
| 111 | 120.60 | 117.88 | 23.36 | 6.45 | 30.09 | 45.74 |
| 112 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 113 | 120.60 | 98.23 | 17.78 | 6.45 | 30.09 | 45.74 |
| 114 | 120.60 | 98.23 | 18.13 | 6.45 | 30.09 | 45.74 |
| 115 | 120.60 | 68.63 | 15.44 | 6.45 | 30.09 | 45.74 |
| 116 | 120.60 | 68.63 | 15.44 | 6.45 | 30.09 | 45.74 |
| 201 | 377.97 | 93.23 | 83.29 | 83.29 | 113.39 | 113.39 |
| 202 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 203 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 204 | 79.65 | 72.01 | 10.99 | 4.60 | 29.14 | 16.61 |
| 205 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 206 | 79.65 | 74.02 | 10.99 | 4.60 | 29.14 | 16.61 |
| 207 | 79.65 | 73.44 | 10.99 | 4.60 | 29.14 | 16.61 |
| 208 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 209 | 48.35 | 43.11 | 2.85 | 2.85 | 14.51 | 14.51 |
| 210 | 79.65 | 72.01 | 10.99 | 4.60 | 29.14 | 16.61 |
| 211 | 120.60 | 54.44 | 23.36 | 6.45 | 30.09 | 45.74 |
| 212 | 142.83 | 141.72 | 16.17 | 16.17 | 42.85 | 42.85 |
| 213 | 120.60 | 98.23 | 18.31 | 6.45 | 30.09 | 45.74 |
| 214 | 120.60 | 98.23 | 19.19 | 6.45 | 30.09 | 45.74 |
| 215 | 120.60 | 68.63 | 15.30 | 6.45 | 30.09 | 45.74 |
| 216 | 120.60 | 68.63 | 15.30 | 6.45 | 30.09 | 45.74 |

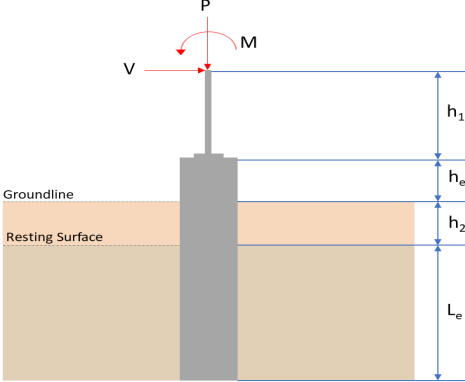
Design Ratio

| Member ID | P | M _z | M _y | V _y | V _z | (P,M _z ,M _y) | Worst LC | KL/r | φ | Status |
|-----------|-------|----------------|----------------|----------------|----------------|-------------------------------------|----------|--------------|--------------|--------|
| 1 | 0.066 | 0.170 | 0.011 | 0.005 | 0.001 | 0.195 | #13 | 0.713 | Not Required | Pass |
| 2 | 0.000 | 0.283 | 0.038 | 0.061 | 0.006 | 0.321 | #13 | 0.034 | Not Required | Pass |
| 3 | 0.001 | 0.497 | 0.010 | 0.050 | 0.001 | 0.505 | #13 | 0.044 | Not Required | Pass |
| 4 | 0.001 | 0.388 | 0.032 | 0.039 | 0.004 | 0.417 | #13 | 0.078 | Not Required | Pass |
| 5 | 0.001 | 0.307 | 0.027 | 0.050 | 0.004 | 0.308 | #13 | 0.073 | Not Required | Pass |
| 6 | 0.002 | 0.562 | 0.022 | 0.058 | 0.003 | 0.584 | #13 | 0.044 | Not Required | Pass |
| 7 | 0.002 | 0.348 | 0.046 | 0.056 | 0.007 | 0.357 | #13 | 0.073 | Not Required | Pass |
| 8 | 0.000 | 0.051 | 0.021 | 0.027 | 0.002 | 0.063 | #13 | 0.088 | Not Required | Pass |
| 9 | 0.002 | 0.063 | 0.014 | 0.002 | 0.000 | 0.078 | #13 | 0.198 | Not Required | Pass |
| 10 | 0.002 | 0.438 | 0.045 | 0.045 | 0.006 | 0.445 | #13 | 0.078 | Not Required | Pass |
| 11 | 0.000 | 0.062 | 0.021 | 0.035 | 0.002 | 0.073 | #13 | 0.088 | Not Required | Pass |
| 12 | 0.000 | 0.340 | 0.042 | 0.070 | 0.008 | 0.382 | #13 | 0.052 | Not Required | Pass |
| 13 | 0.001 | 0.152 | 0.044 | 0.046 | 0.002 | 0.164 | #13 | 0.265 | Not Required | Pass |
| 14 | 0.001 | 0.122 | 0.044 | 0.035 | 0.002 | 0.135 | #13 | 0.177 | Not Required | Pass |
| 15 | 0.000 | 0.054 | 0.014 | 0.022 | 0.001 | 0.065 | #13 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.042 | 0.014 | 0.017 | 0.001 | 0.054 | #13 | Not Required | Not Required | Pass |
| 101 | 0.077 | 0.180 | 0.000 | 0.006 | 0.000 | 0.210 | #13 | 0.713 | Not Required | Pass |
| 102 | 0.001 | 0.364 | 0.046 | 0.077 | 0.008 | 0.410 | #13 | 0.052 | Not Required | Pass |
| 103 | 0.002 | 0.615 | 0.018 | 0.062 | 0.003 | 0.629 | #13 | 0.044 | Not Required | Pass |
| 104 | 0.002 | 0.489 | 0.045 | 0.050 | 0.005 | 0.519 | #13 | 0.078 | Not Required | Pass |
| 105 | 0.002 | 0.381 | 0.047 | 0.062 | 0.007 | 0.393 | #13 | 0.073 | Not Required | Pass |
| 106 | 0.002 | 0.615 | 0.018 | 0.062 | 0.003 | 0.629 | #13 | 0.044 | Not Required | Pass |
| 107 | 0.002 | 0.381 | 0.047 | 0.062 | 0.007 | 0.393 | #13 | 0.073 | Not Required | Pass |
| 108 | 0.000 | 0.037 | 0.022 | 0.030 | 0.002 | 0.051 | #13 | 0.088 | Not Required | Pass |
| 109 | 0.002 | 0.061 | 0.012 | 0.001 | 0.000 | 0.075 | #13 | 0.198 | Not Required | Pass |
| 110 | 0.002 | 0.489 | 0.045 | 0.050 | 0.005 | 0.519 | #13 | 0.078 | Not Required | Pass |
| 111 | 0.000 | 0.053 | 0.022 | 0.037 | 0.002 | 0.054 | #13 | 0.088 | Not Required | Pass |
| 112 | 0.001 | 0.364 | 0.046 | 0.077 | 0.008 | 0.410 | #13 | 0.052 | Not Required | Pass |
| 113 | 0.001 | 0.168 | 0.045 | 0.047 | 0.002 | 0.204 | #13 | 0.265 | Not Required | Pass |
| 114 | 0.001 | 0.148 | 0.045 | 0.038 | 0.002 | 0.180 | #13 | 0.265 | Not Required | Pass |
| 115 | 0.001 | 0.178 | 0.022 | 0.037 | 0.002 | 0.199 | #13 | 0.439 | Not Required | Pass |
| 116 | 0.001 | 0.139 | 0.023 | 0.030 | 0.002 | 0.157 | #13 | 0.439 | Not Required | Pass |
| 201 | 0.066 | 0.170 | 0.011 | 0.005 | 0.001 | 0.195 | #13 | 0.713 | Not Required | Pass |
| 202 | 0.000 | 0.340 | 0.042 | 0.070 | 0.008 | 0.382 | #13 | 0.052 | Not Required | Pass |
| 203 | 0.002 | 0.562 | 0.022 | 0.058 | 0.003 | 0.584 | #13 | 0.044 | Not Required | Pass |
| 204 | 0.002 | 0.438 | 0.045 | 0.045 | 0.006 | 0.445 | #13 | 0.078 | Not Required | Pass |
| 205 | 0.002 | 0.348 | 0.046 | 0.056 | 0.007 | 0.357 | #13 | 0.073 | Not Required | Pass |
| 206 | 0.001 | 0.497 | 0.010 | 0.050 | 0.001 | 0.505 | #13 | 0.044 | Not Required | Pass |
| 207 | 0.001 | 0.307 | 0.027 | 0.050 | 0.004 | 0.308 | #13 | 0.073 | Not Required | Pass |
| 208 | 0.000 | 0.042 | 0.014 | 0.017 | 0.001 | 0.054 | #13 | Not Required | Not Required | Pass |
| 209 | 0.002 | 0.063 | 0.014 | 0.002 | 0.000 | 0.078 | #13 | 0.198 | Not Required | Pass |
| 210 | 0.001 | 0.388 | 0.032 | 0.039 | 0.004 | 0.417 | #13 | 0.078 | Not Required | Pass |
| 211 | 0.000 | 0.054 | 0.014 | 0.022 | 0.001 | 0.065 | #13 | Not Required | Not Required | Pass |
| 212 | 0.000 | 0.283 | 0.038 | 0.061 | 0.006 | 0.321 | #13 | 0.034 | Not Required | Pass |
| 213 | 0.001 | 0.152 | 0.044 | 0.046 | 0.002 | 0.164 | #13 | 0.177 | Not Required | Pass |
| 214 | 0.001 | 0.122 | 0.044 | 0.035 | 0.002 | 0.135 | #13 | 0.265 | Not Required | Pass |
| 215 | 0.001 | 0.182 | 0.022 | 0.035 | 0.002 | 0.200 | #13 | 0.439 | Not Required | Pass |
| 216 | 0.001 | 0.142 | 0.022 | 0.027 | 0.002 | 0.162 | #13 | 0.439 | Not Required | Pass |

Definitions

Φ_t Safety factor for tensile

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

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|-------|-------------|--------|--------|-------------|-------|-------|---------------|-------|-------|---------------|-------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 6.75$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.385</td> <td>6.185</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.358</td> <td>-0.599</td> </tr> <tr> <td>V_z (kip)</td> <td>0.056</td> <td>0.081</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.292</td> <td>0.426</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.296</td> <td>14.158</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ 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) | 4.385 | 6.185 | V_x (kip) | -0.358 | -0.599 | V_z (kip) | 0.056 | 0.081 | M_x (kipft) | 0.292 | 0.426 | M_z (kipft) | 8.296 | 14.158 | |
| 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) | 4.385 | 6.185 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.358 | -0.599 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.056 | 0.081 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.292 | 0.426 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 8.296 | 14.158 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.358 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.11933 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(8.296 \text{ kipft}) + ((-0.358 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.7653 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.4983 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.056 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.018667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.292 \text{ kipft}) + ((0.056 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.097333 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.5577 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.4983 \text{ ft}), (2.5577 \text{ ft})]$$

$$L_{e,req} = 6.498 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.498 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.96267$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.385 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.62035 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62035 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.31018$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.11933 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7653 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7653 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.7653 \text{ kipft/ft})) + (4 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5915 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.7653 \text{ kipft/ft})) + (3 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.7653 \text{ kipft/ft})) + (2 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.28903 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.7653 \text{ kipft/ft})) + ((-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.97744 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5915 \text{ ft})}{2}$$

$$p_a = 0.34436 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28903 \text{ kip/ft}^2)}{(0.34436 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.83934$$

Status: **PASS**
Ratio: **0.840**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.97744 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96537$$

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

Considering z-direction:

$H_o = 0.018667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.097333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.097333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.018667 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.097333 \text{ kipft/ft})) + (4 \times (0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7606 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.097333 \text{ kipft/ft})) + (3 \times (0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.097333 \text{ kipft/ft})) + (2 \times (0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.027984 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.097333 \text{ kipft/ft})) + ((0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.066333 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7606 \text{ ft})}{2}$$

$$p_a = 0.35704 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.027984 \text{ kip/ft}^2)}{(0.35704 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.078377$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

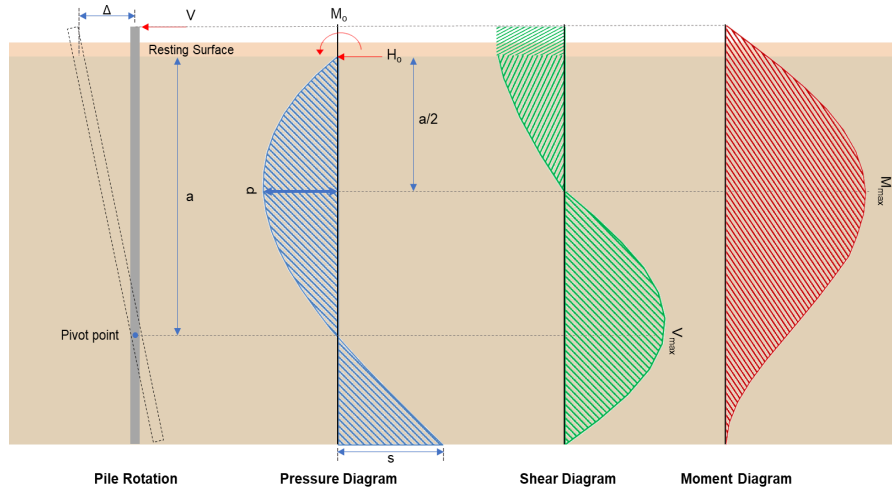
Status: **PASS**
Ratio: **0.080**

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

$$Ratio = \frac{(0.066333 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.065514$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.599 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.19967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.158 \text{ kipft}) + ((-0.599 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.7193 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7193 \text{ kipft/ft})}{(-0.19967 \text{ kip/ft})}$$

$$E = 23.636 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7193 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.19967 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.7193 \text{ kipft/ft})) + (4 \times (-0.19967 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.59 \text{ ft}$$

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

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

$$V_{max} = ((-0.19967 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.59 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.59 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.1114 \text{ kip}$$

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

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

$$M_{max} = ((-0.19967 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(23.636 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.506 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = 0.027 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.426 \text{ kipft}) + ((0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.142 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.142 \text{ kipft/ft})}{(0.027 \text{ kip/ft})}$$

$$E = 5.2593 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.142 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (0.027 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.142 \text{ kipft/ft})) + (4 \times (0.027 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7594 \text{ ft}$$

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

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

$$V_{max} = ((0.027 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7594 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7594 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.16531 \text{ kip}$$

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

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

$$M_{max} = ((0.027 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(5.2593 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.50885 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.185 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.985 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.985 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

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

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

Status: **PASS**
Ratio: **1.000**

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (3 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1492.5 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.185 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0041441$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 203.86 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.185 \text{ kip} \rightarrow 6185 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6185 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.593 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 237.06 \text{ kip}$$

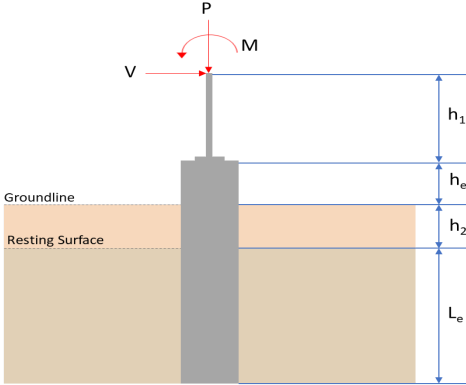
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.593 \text{ kip}), (237.06 \text{ kip})]$$

| | | |
|---|---|---|
| <p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p> | <p style="text-align: center;">$V_c = 82.593 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.593 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.496 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.1114 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.1114 \text{ kip})}{(78.496 \text{ kip})}$ $Ratio = 0.052377$ <p>Considering z-direction:</p> <p>$V_{max} = 0.16531 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.16531 \text{ kip})}{(78.496 \text{ kip})}$ $Ratio = 0.002106$ | <p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ | |

| | | |
|------------------|---|---|
| <p>14.5.2.1b</p> | <p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 13.506 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(13.506 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.19877$ | <p>Status: PASS Ratio: 0.200</p> |
| | <p>Considering z-direction: $M_{max} = 0.50885 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.50885 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.007489$ | <p>Status: PASS Ratio: 0.010</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|-------|-------------|--------|--------|-------------|--------|--------|---------------|--------|--------|---------------|-------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 6.75$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>4.385</td> <td>6.185</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.358</td> <td>-0.599</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.056</td> <td>-0.081</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.292</td> <td>-0.426</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.296</td> <td>14.158</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ 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) | 4.385 | 6.185 | V_x (kip) | -0.358 | -0.599 | V_z (kip) | -0.056 | -0.081 | M_x (kipft) | -0.292 | -0.426 | M_z (kipft) | 8.296 | 14.158 | |
| 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) | 4.385 | 6.185 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.358 | -0.599 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | -0.056 | -0.081 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | -0.292 | -0.426 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 8.296 | 14.158 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.358 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.11933 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_x + (V_x H)}{D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(8.296 \text{ kipft}) + ((-0.358 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 2.7653 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,x} = 6.4983 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.056 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.018667 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.292 \text{ kipft}) + ((-0.056 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.097333 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.0506 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(6.4983 \text{ ft}), (2.0506 \text{ ft})]$$

$$L_{e,req} = 6.498 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(6.498 \text{ ft})}{(6.75 \text{ ft})}$$

$$\text{Ratio} = 0.96267$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

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

$$A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$$

$$A = 7.0686 \text{ ft}^2$$

q - End-bearing pressure

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

$$q = \frac{(4.385 \text{ kip})}{(7.0686 \text{ ft}^2)}$$

$$q = 0.62035 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.62035 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.31018$$

Status: **PASS**
Ratio: **0.310**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(6.75 \text{ ft})}{(36 \text{ in})}$$

$$L/D = 2.25$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.11933 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 2.7653 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.7653 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (2.7653 \text{ kipft/ft})) + (4 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.5915 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.7653 \text{ kipft/ft})) + (3 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (2.7653 \text{ kipft/ft})) + (2 \times (-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.28903 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.7653 \text{ kipft/ft})) + ((-0.11933 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.97744 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.5915 \text{ ft})}{2}$$

$$p_a = 0.34436 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.28903 \text{ kip/ft}^2)}{(0.34436 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.83934$$

Status: **PASS**
Ratio: **0.840**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.97744 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.96537$$

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

Considering z-direction:

$H_o = -0.018667 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.097333 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.097333 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.097333 \text{ kipft/ft})) + (4 \times (-0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7606 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (0.097333 \text{ kipft/ft})) + (3 \times (-0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]^2}{(6.75 \text{ ft})^2 \times [(3 \times (0.097333 \text{ kipft/ft})) + (2 \times (-0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]}$$

$$p = 0.000083022 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (0.097333 \text{ kipft/ft})) + ((-0.018667 \text{ kip/ft}) \times (6.75 \text{ ft}))]}{(6.75 \text{ ft})^2}$$

$$s = 0.014204 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7606 \text{ ft})}{2}$$

$$p_a = 0.35704 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.000083022 \text{ kip/ft}^2)}{(0.35704 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.00023253$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (6.75 \text{ ft})$$

$$p_s = 1.0125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

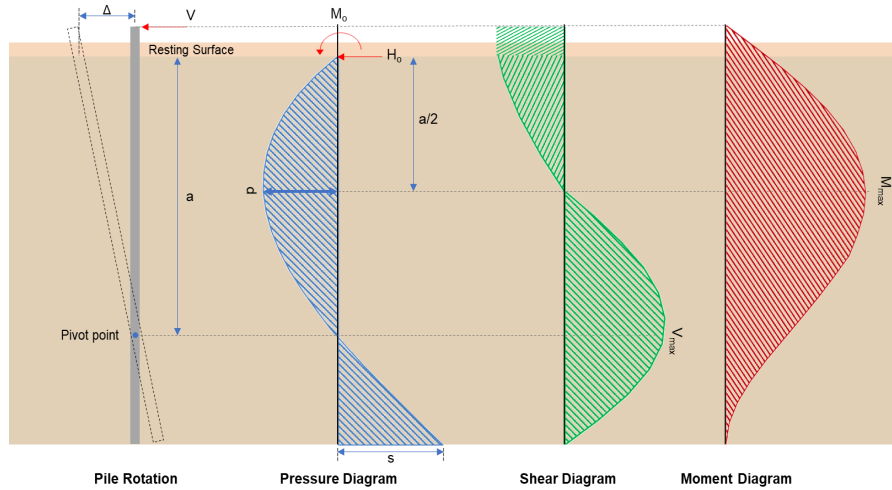
Status: **PASS**
Ratio: **0.000**

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

$$Ratio = \frac{(0.014204 \text{ kip/ft}^2)}{(1.0125 \text{ kip/ft}^2)}$$

$$Ratio = 0.014029$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.599 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.19967 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.158 \text{ kipft}) + ((-0.599 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.7193 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.7193 \text{ kipft/ft})}{(-0.19967 \text{ kip/ft})}$$

$$E = 23.636 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.7193 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.19967 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (4.7193 \text{ kipft/ft})) + (4 \times (-0.19967 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.59 \text{ ft}$$

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

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

$$V_{max} = ((-0.19967 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.59 \text{ ft})}{(6.75 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.59 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.1114 \text{ kip}$$

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

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

$$M_{max} = ((-0.19967 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(23.636 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (23.636 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.59 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 13.506 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.081 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.027 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.426 \text{ kipft}) + ((-0.081 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 0.142 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.142 \text{ kipft/ft})}{(-0.027 \text{ kip/ft})}$$

$$E = 5.2593 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.142 \text{ kipft/ft}) \times (6.75 \text{ ft})) + (3 \times (-0.027 \text{ kip/ft}) \times (6.75 \text{ ft})^2)}{(6 \times (0.142 \text{ kipft/ft})) + (4 \times (-0.027 \text{ kip/ft}) \times (6.75 \text{ ft}))}$$

$$a = 4.7594 \text{ ft}$$

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

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

$$V_{max} = ((-0.027 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7594 \text{ ft})}{(6.75 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7594 \text{ ft})}{(6.75 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.16531 \text{ kip}$$

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

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

$$M_{max} = ((-0.027 \text{ kip/ft}) \times (36 \text{ in}) \times (6.75 \text{ ft})) \times \left[\left(\frac{(5.2593 \text{ ft})}{(6.75 \text{ ft})} + \frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right) - \left[\left(\frac{4 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 3 \right) \times \left(\frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (5.2593 \text{ ft})}{(6.75 \text{ ft})} + 2 \right) \times \left(\frac{(4.7594 \text{ ft})}{2 \times (6.75 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.50885 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(6.185 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.985 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.985 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$\text{Ratio} = 0.99533$$

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

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

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

Status: **PASS**
Ratio: **1.000**

Main reinforcement: **6 - #5 (0.625 in)**
Ties: **#3(0.375 in) - 10 in**

Axial Compression Strength (ACI 318-19, LFRD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (3 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1492.5 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(6.185 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0041441$$

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

Shear Strength (ACI 318-19, LFRD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 203.86 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 6.185 \text{ kip} \rightarrow 6185 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(6185 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.593 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 237.06 \text{ kip}$$

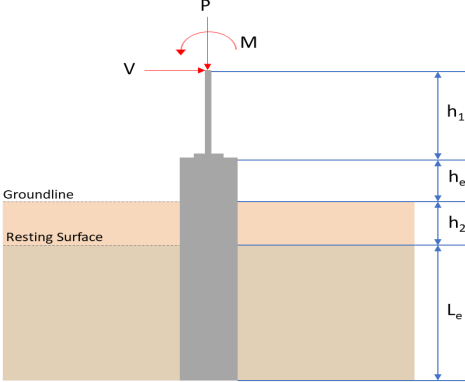
V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.593 \text{ kip}), (237.06 \text{ kip})]$$

| | | |
|---|---|---|
| <p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p> | <p style="text-align: center;">$V_c = 82.593 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.593 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.496 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.1114 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.1114 \text{ kip})}{(78.496 \text{ kip})}$ $Ratio = 0.052377$ <p>Considering z-direction:</p> <p>$V_{max} = 0.16531 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(0.16531 \text{ kip})}{(78.496 \text{ kip})}$ $Ratio = 0.002106$ | <p>Status: PASS Ratio: 0.050</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ | |

| | | |
|------------------|---|---|
| <p>14.5.2.1b</p> | <p style="text-align: center;">$S_m = 4580.4 \text{ in}^3$</p> <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{3 \text{ ksi}} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_{ck} \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$ $\phi M_{n,2} = 632.67 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$ $\phi M_n = 67.947 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 13.506 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(13.506 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.19877$ | <p>Status: PASS Ratio: 0.200</p> |
| | <p>Considering z-direction: $M_{max} = 0.50885 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.50885 \text{ kipft})}{(67.947 \text{ kipft})}$ $\text{Ratio} = 0.007489$ | <p>Status: PASS Ratio: 0.010</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|---|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|-------|-------------|--------|--------|-------------|-------|-------|---------------|-------|-------|---------------|-------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: round $D = 36$ in - Pile diameter $L = 7$ 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</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1079 1193 1171"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>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 1265 935 1435"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.044</td> <td>7.137</td> </tr> <tr> <td>V_x (kip)</td> <td>-0.393</td> <td>-0.652</td> </tr> <tr> <td>V_z (kip)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.000</td> <td>0.000</td> </tr> <tr> <td>M_z (kipft)</td> <td>8.778</td> <td>14.982</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 3$ 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) | 5.044 | 7.137 | V_x (kip) | -0.393 | -0.652 | V_z (kip) | 0.000 | 0.000 | M_x (kipft) | 0.000 | 0.000 | M_z (kipft) | 8.778 | 14.982 | |
| 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) | 5.044 | 7.137 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -0.393 | -0.652 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.000 | 0.000 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.000 | 0.000 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 8.778 | 14.982 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{D}$ $H_o = \frac{(-0.393 \text{ kip})}{(36 \text{ in})}$ $H_o = -0.131 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

| | | |
|----------|--|---|
| | $M_o = \frac{(8.778 \text{ kipft}) + ((-0.393 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$ $M_o = 2.926 \text{ kipft/ft}$ <p>Required depth of embedment in earth:</p> $L_x^3 - \left(14.14 \times \frac{H_o \times L_x}{R}\right) - \left(18.85 \times \frac{M_o}{R}\right) = 0$ <p>Solving the cubic equation: $L_{e,x} = 6.5909 \text{ ft}$ - Required depth in x-direction,</p> <p>Considering z-direction: $L_{e,z} = 0 \text{ ft}$ - Required depth in z-direction,</p> <p>Minimum embedded depth required: $L_{e,req}$ - Depth of pile required,</p> $L_{e,req} = \text{MAX}[L_{e,x}, L_{e,z}]$ $L_{e,req} = \text{MAX}[(6.5909 \text{ ft}), (0 \text{ ft})]$ $L_{e,req} = 6.591 \text{ ft}$ <p>L_e - Actual embedded length of pile,</p> $L_e = L - h_c - h_2$ $L_e = (7 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$ $L_e = 7 \text{ ft}$ <p>Ratio - Embedded depth</p> $\text{Ratio} = \frac{L_{e,req}}{L_e}$ $\text{Ratio} = \frac{(6.591 \text{ ft})}{(7 \text{ ft})}$ $\text{Ratio} = 0.94157$ | <p>Status: PASS Ratio: 0.940</p> |
| | <p>End-bearing Capacity (ASD)</p> <p>A - Pile cross-section area</p> $A = \pi \left(\frac{D}{2}\right)^2$ $A = \pi \times \left(\frac{(36 \text{ in})}{2}\right)^2$ $A = 7.0686 \text{ ft}^2$ <p>q - End-bearing pressure</p> $q = \frac{P_c}{A}$ $q = \frac{(5.044 \text{ kip})}{(7.0686 \text{ ft}^2)}$ $q = 0.71358 \text{ kip/ft}^2$ <p>Check bearing capacity ratio:</p> <p>Ratio - Capacity</p> $\text{Ratio} = \frac{q}{q_a}$ $\text{Ratio} = \frac{(0.71358 \text{ kip/ft}^2)}{(2000 \text{ psf})}$ $\text{Ratio} = 0.35679$ | <p>Status: PASS Ratio: 0.360</p> |
| Czerniak | <p>Lateral Soil Pressure (ASD):</p> <p>L/D - Length to least lateral dimension ratio,</p> $L/D = \frac{L}{D}$ $L/D = \frac{(7 \text{ ft})}{(36 \text{ in})}$ | |

$$L/D = 2.3333$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.131$ kip/ft - Lateral force per length of pile,

$M_o = 2.926$ kipft/ft - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (2.926 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.131 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (2.926 \text{ kipft/ft})) + (4 \times (-0.131 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7675 \text{ ft}$$

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

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

$$p = \frac{1.178 \times [(4 \times (2.926 \text{ kipft/ft})) + (3 \times (-0.131 \text{ kip/ft}) \times (7 \text{ ft}))]^2}{(7 \text{ ft})^2 \times [(3 \times (2.926 \text{ kipft/ft})) + (2 \times (-0.131 \text{ kip/ft}) \times (7 \text{ ft}))]}$$

$$p = 0.27751 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{9.425 \times [(2 \times (2.926 \text{ kipft/ft})) + ((-0.131 \text{ kip/ft}) \times (7 \text{ ft}))]}{(7 \text{ ft})^2}$$

$$s = 0.94923 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.7675 \text{ ft})}{2}$$

$$p_a = 0.35756 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.27751 \text{ kip/ft}^2)}{(0.35756 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.77612$$

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (7 \text{ ft})$$

$$p_s = 1.05 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

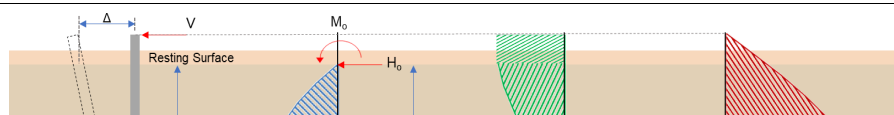
$$\text{Ratio} = \frac{s}{p_s}$$

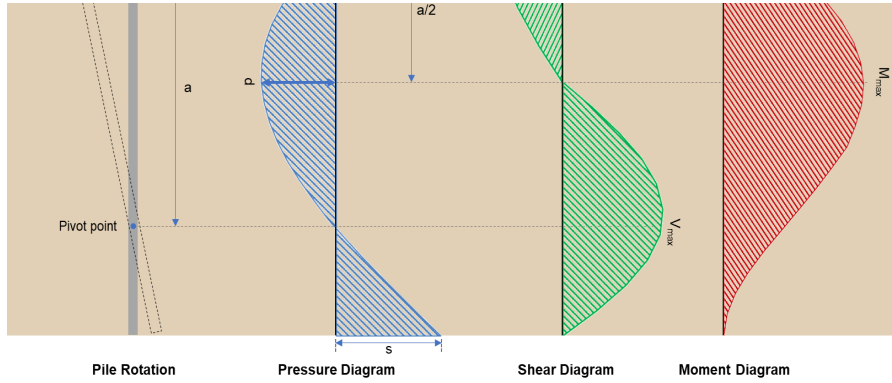
$$\text{Ratio} = \frac{(0.94923 \text{ kip/ft}^2)}{(1.05 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.90403$$

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

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





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.652 \text{ kip})}{(36 \text{ in})}$$

$$H_o = -0.21733 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(14.982 \text{ kipft}) + ((-0.652 \text{ kip}) \times (0 \text{ ft}))}{(36 \text{ in})}$$

$$M_o = 4.994 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(4.994 \text{ kipft/ft})}{(-0.21733 \text{ kip/ft})}$$

$$E = 22.979 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (4.994 \text{ kipft/ft}) \times (7 \text{ ft})) + (3 \times (-0.21733 \text{ kip/ft}) \times (7 \text{ ft})^2)}{(6 \times (4.994 \text{ kipft/ft})) + (4 \times (-0.21733 \text{ kip/ft}) \times (7 \text{ ft}))}$$

$$a = 4.7651 \text{ ft}$$

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

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

$$V_{max} = ((-0.21733 \text{ kip/ft}) \times (36 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (22.979 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7651 \text{ ft})}{(7 \text{ ft})} \right)^2 + 4 \times \left(\frac{3 \times (22.979 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7651 \text{ ft})}{(7 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 4.2216 \text{ kip}$$

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

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

$$M_{max} = ((-0.21733 \text{ kip/ft}) \times (36 \text{ in}) \times (7 \text{ ft})) \times \left[\left(\frac{(22.979 \text{ ft})}{(7 \text{ ft})} + \frac{(4.7651 \text{ ft})}{2 \times (7 \text{ ft})} \right) - \left[\left(\frac{4 \times (22.979 \text{ ft})}{(7 \text{ ft})} + 3 \right) \times \left(\frac{(4.7651 \text{ ft})}{2 \times (7 \text{ ft})} \right)^3 + \left(\frac{3 \times (22.979 \text{ ft})}{(7 \text{ ft})} + 2 \right) \times \left(\frac{(4.7651 \text{ ft})}{2 \times (7 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 14.358 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

$f'_{ck} = 3 \text{ ksi}$ - Concrete strength,
 $f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
 $\phi = 0.65$ - Reduction factor for axial strength,
 $\alpha = 0.85$ - Alpha factor for axial strength,
 $A_g = 1017.9 \text{ in}^2$ - 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

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(7.137 \text{ kip})}{(0.65) \times (0.85)} - (0.85 \times (3 \text{ ksi}) \times (1017.9 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (1017.9 \text{ in}^2)) \right]$$

$$A_{st,required} = -44.955 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-44.955 \text{ in}^2), (0.0018 \times (1017.9 \text{ in}^2))]$$

$$A_{min} = 1.8322 \text{ in}^2$$

n_{rebar} - Required number of reinforcement,

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

$$n_{rebar} = \frac{(1.8322 \text{ in}^2)}{(0.3068 \text{ in}^2)}$$

$$n_{rebar} = 6$$

A_{st} - Actual total reinforcement area,

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

$$A_{st} = (6) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$$

$$A_{st} = 1.8408 \text{ in}^2$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(1.8322 \text{ in}^2)}{(1.8408 \text{ in}^2)}$$

$$\text{Ratio} = 0.99533$$

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. 10Ø: Use #3(0.375 in)

25.7.2.1

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

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

$$s_{ties} = \text{Min} [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), (36 \text{ in})]$$

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

Summary:

Status: **PASS**
Ratio: **1.000**

Main reinforcement: **6 - #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

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

$$\phi P_N = (0.65) \times 0.85 \times \left[(0.85 \times (3 \text{ ksi}) \times [(1017.9 \text{ in}^2) - (1.8408 \text{ in}^2)]) + ((60 \text{ ksi}) \times (1.8408 \text{ in}^2)) \right]$$

$$\phi P_N = 1492.5 \text{ kip}$$

Ratio - Capacity

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

$$\text{Ratio} = \frac{(7.137 \text{ kip})}{(1492.5 \text{ kip})}$$

$$\text{Ratio} = 0.0047819$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

$b_w = 36 \text{ in}$ - Effective width,
 d - Effective depth

$$d = 0.80 D$$

$$d = 0.80 \times (36 \text{ in})$$

$$d = 28.8 \text{ in}$$

22.5.5.1.3

λ_s - size effect modification factor

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

$$\lambda_s = \text{MIN} \left[\sqrt{\frac{2}{1 + \frac{(28.8 \text{ in})}{10}}}, 1 \right]$$

$$\lambda_s = 0.71796$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.1

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

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

$$V_{c,max} = 5 \times (0.71796) \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,max} = 203.86 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 7.137 \text{ kip} \rightarrow 7137 \text{ lbf}$.

22.5.5.1.1(a)

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

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

$$V_{c,a} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + \frac{(7137 \text{ lbf})}{6 \times (1017.9 \text{ in}^2)} \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,a} = 82.754 \text{ kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.

22.5.5.1.2

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

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

$$V_{c,b} = \left[2 \times (0.71796) \times \sqrt{(3000 \text{ psi})} + (0.05 \times (3000 \text{ psi})) \right] \times (36 \text{ in}) \times (28.8 \text{ in})$$

$$V_{c,b} = 237.06 \text{ kip}$$

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(203.86 \text{ kip}), (82.754 \text{ kip}), (237.06 \text{ kip})]$$

| | | |
|---|---|---|
| <p>22.5.1.2</p> <p>22.5.8.5.3</p> <p>22.5.1.1</p> | <p style="text-align: center;">$V_c = 82.754 \text{ kip}$</p> <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (36 \text{ in}) \times (28.8 \text{ in})$ $V_{s,a} = 454.3 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>$V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yw} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (28.8 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 38.17 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = MIN[V_{s,a}, V_{s,b}]$ $V_s = MIN[(454.3 \text{ kip}), (38.17 \text{ kip})]$ $V_s = 38.17 \text{ kip}$ <p>ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((82.754 \text{ kip}) + (38.17 \text{ kip}))$ $\phi V_n = 78.601 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 4.2216 \text{ kip}$ - Maximum shear force in the x-direction, <i>Ratio</i> - Capacity</p> $Ratio = \frac{V_{max}}{\phi V_n}$ $Ratio = \frac{(4.2216 \text{ kip})}{(78.601 \text{ kip})}$ $Ratio = 0.05371$ | <p>Status: PASS Ratio: 0.050</p> |
| <p>14.5.2.1b</p> | <p>Flexural Strength (ACI 318-19, LFRD)</p> <p>S_m - Section modulus</p> $S_m = \frac{\pi D^3}{32}$ $S_m = \frac{\pi \times (36 \text{ in})^3}{32}$ $S_m = 4580.4 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of:</p> <p>$\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 4580.442 \text{ in}^3$ $\phi M_{n,1} = 67.947 \text{ kipft}$ <p>$\phi M_{n,2}$</p> | |

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

$$\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (4580.4 \text{ in}^3)$$

$$\phi M_{n,2} = 632.67 \text{ kipft}$$

Therefore,

ϕM_n - Allowable flexural strength,

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

$$\phi M_n = \text{MIN}[(67.947 \text{ kipft}), (632.67 \text{ kipft})]$$

$$\phi M_n = 67.947 \text{ kipft}$$

Considering x-direction:

$M_{max} = 14.358 \text{ kipft}$ - Maximum moment in the x-direction,

$Ratio$ - Capacity

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

$$Ratio = \frac{(14.358 \text{ kipft})}{(67.947 \text{ kipft})}$$

$$Ratio = 0.21132$$

Status: **PASS**
Ratio: **0.210**