

Your Project Calculations



Project Name: MTSOLAR_D6316KI3543B

S3D Model Link:

https://platform.skyciv.com/structural?preload_name=MTSOLAR_D6316KI3543B&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/4_2023

Public Model Link:

https://platform.skyciv.com/structural-viewer?project_id=rKlrwRxUSMC4aRAAjfnD6MzQLOloPZUP8I2H1NZtjUR8JeDUQobBggMGkIBWjjNi

Array Specification

| | |
|-----------------------------|------------------------------|
| Product: | Beam |
| Unique ID: | 1P-0-6TOP-SD-57-L-5Hx3W-J8EG |
| Duty Classification: | SD |
| Module Width: | 40.60 in |
| Module Length: | 72.40in |
| Number of Rows: | 5 |
| Number of Columns: | 3 |
| Total Number of Modules: | 15 |
| Desired Tilt Angle: | 0 |
| Front Edge Clearance: | 11 |
| Total Array Height at Tilt: | 11.00 ft |
| Total Frame Length: | 17.00 ft |
| Frame Weight: | 667 lbs |
| Array Dimensions N/S: | 17.13 ft |
| Array Dimensions E/W: | 18.35 ft |
| Rail Length: | 205.50 in |
| Rail Spacing: | 3.02 ft |
| Rail Check: | Not Checked |

Support Specifications

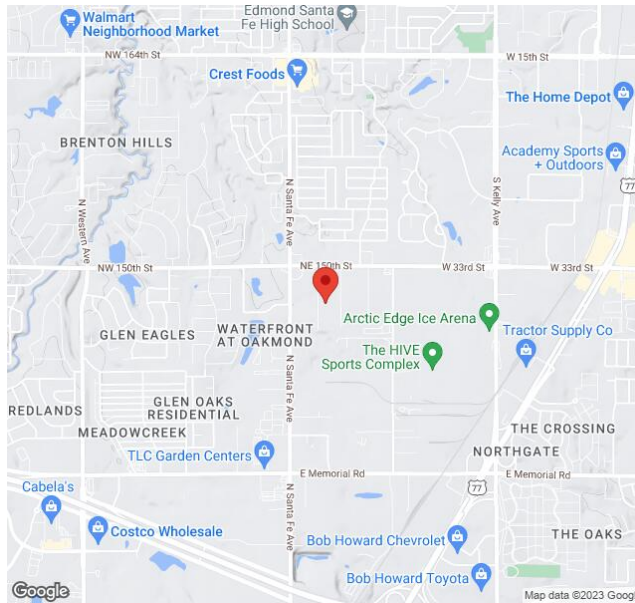
| | |
|--------------------------|-----------------|
| Pole Size: | 6in Pipe Sch 40 |
| Pole Length above Grade: | 11.00 ft |
| Number of Poles: | 1 |
| Pole Spacing: | 0 |

Foundation Specifications

| | |
|---------------------------------|----------------------|
| Foundation Type: | Square |
| Foundation Dimensions: | 48 x 48 in |
| Foundation Depth (below grade): | Pile 1: 4.25 ft |
| Foundation Volume: | 2.519 y ³ |
| Foundation Result: | PASSED |
| Mount Twist: | 0.000000 kip |

Site Info

| | |
|----------------------------|---|
| Risk Category: | I |
| Exposure: | C |
| Soil Classification: | sand |
| Site Location: | 14800 Santa Fe Crossing Dr, Edmond, OK 73013, USA |
| Wind Speed: | 100 mph |
| Snow Load: | 10 psf |
| Design Uplift Pressure: | Multiple pressures |
| Design Downforce Pressure: | Multiple pressures |
| Design Snow Pressure: | 0.011048 ksf |



Design Disclaimer

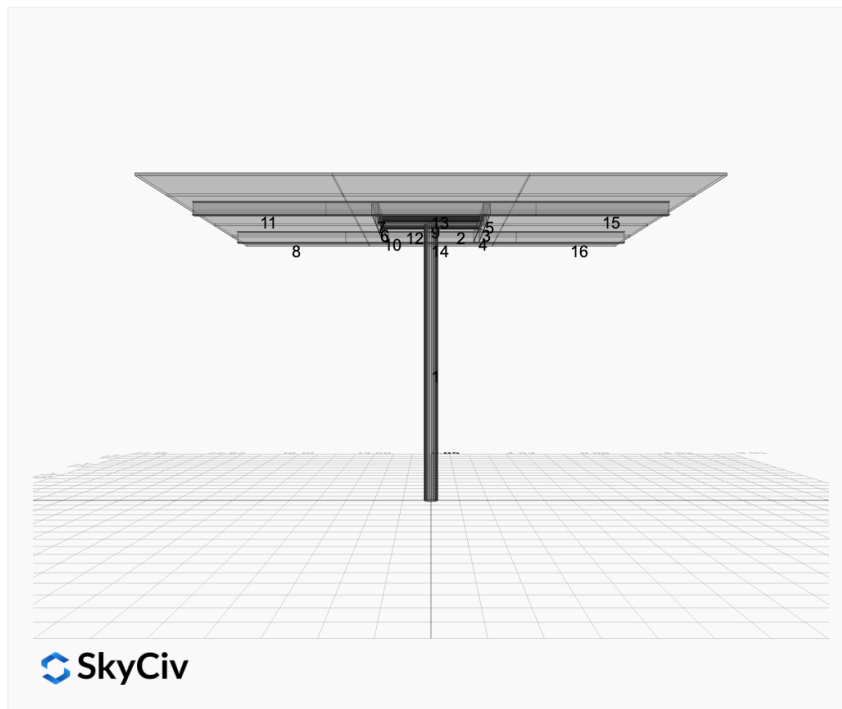
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

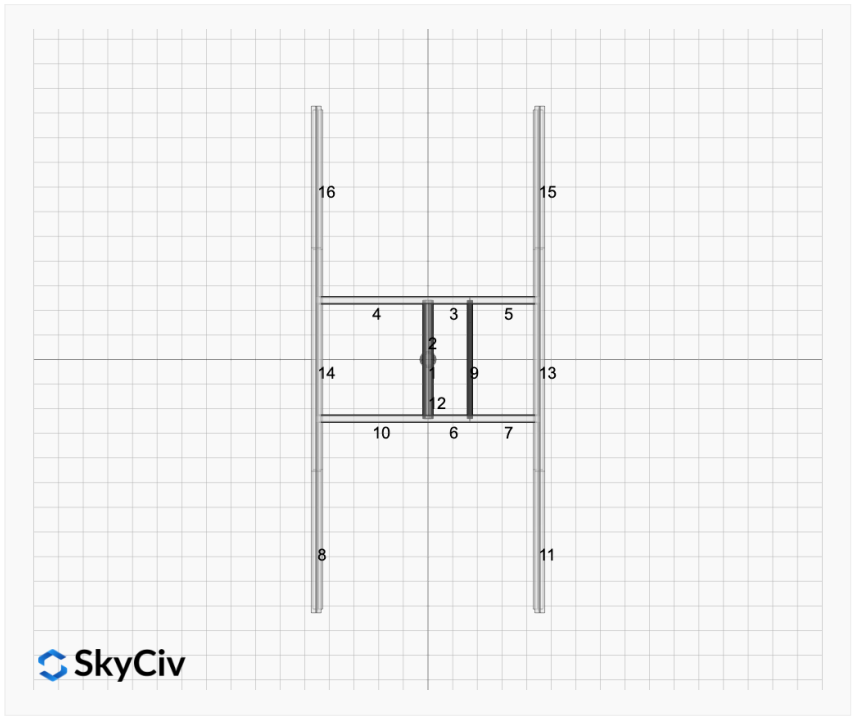
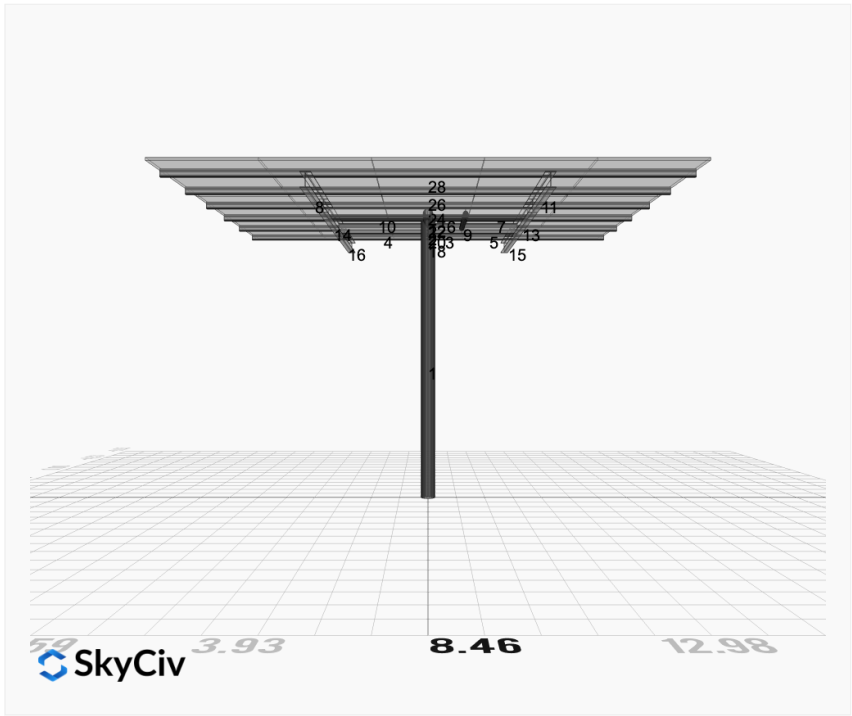
AutoDesigner Input

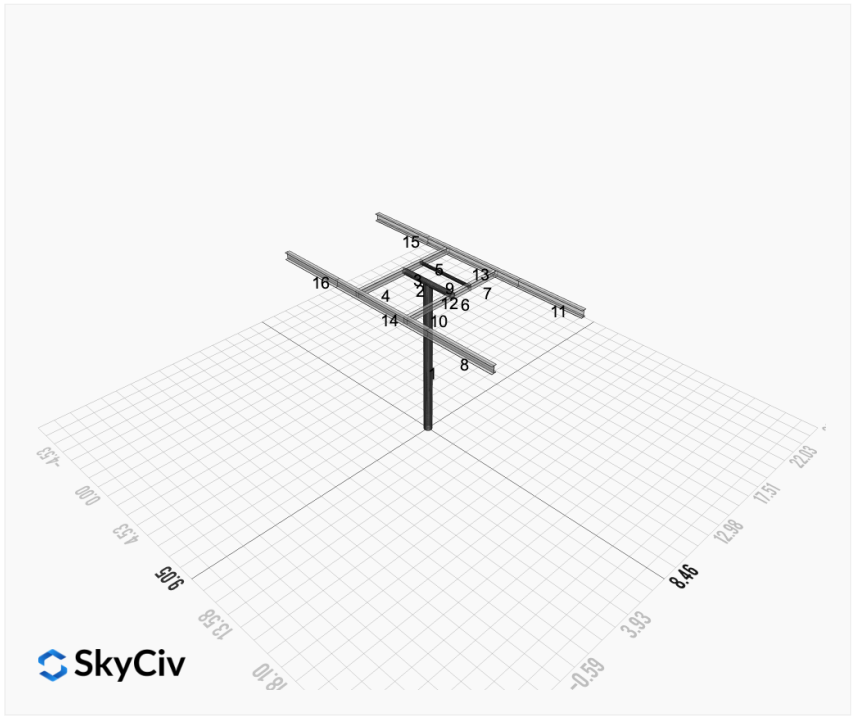
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Design Notes:

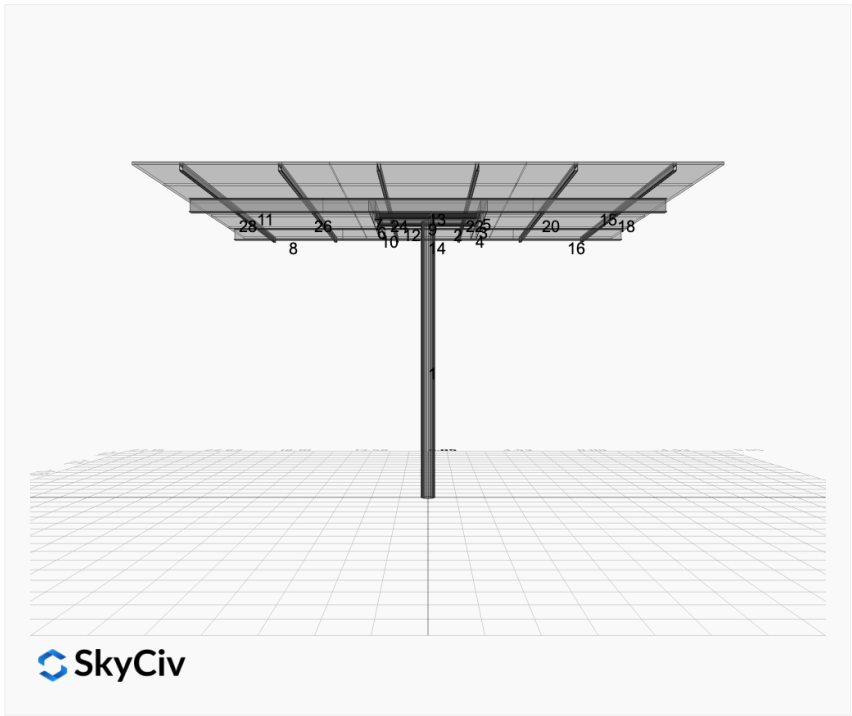
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only





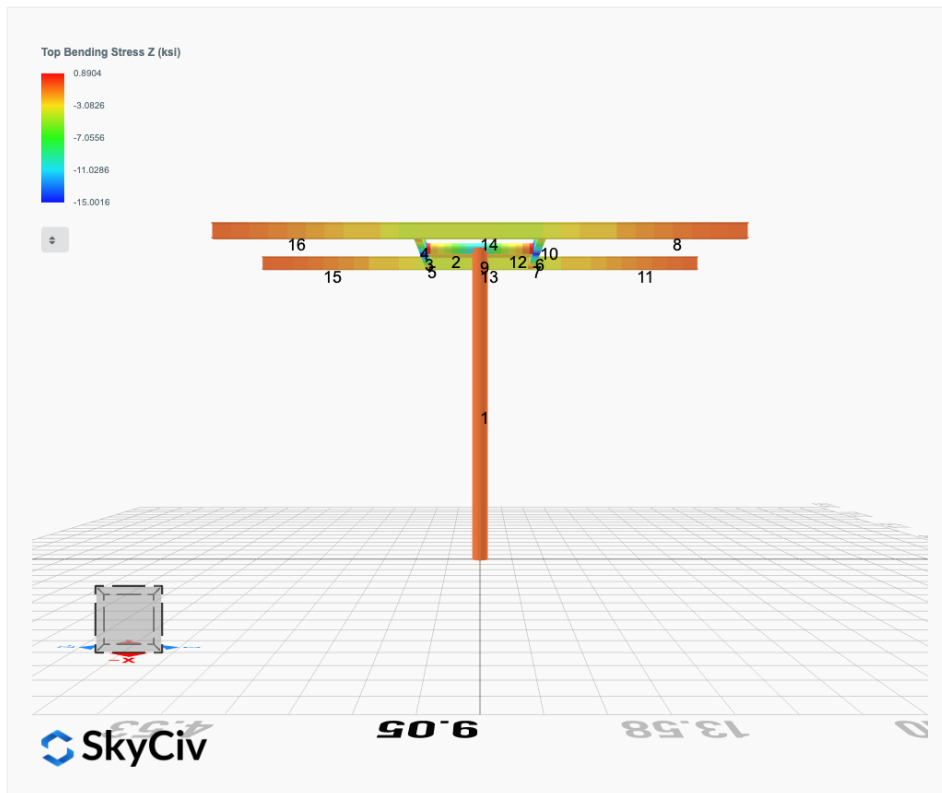
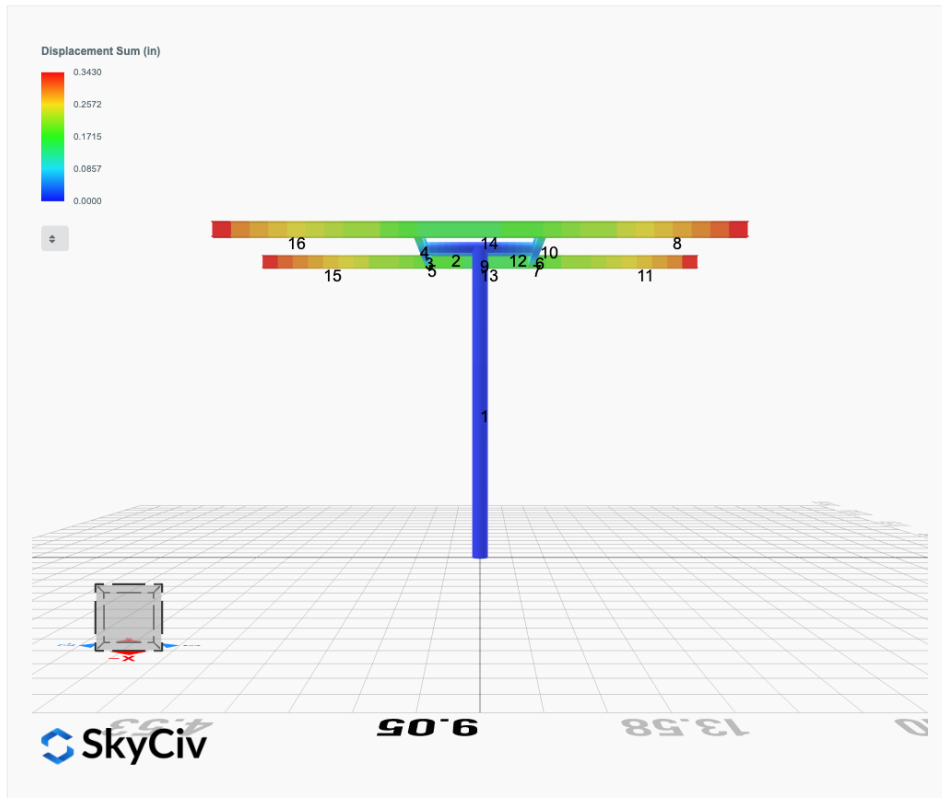


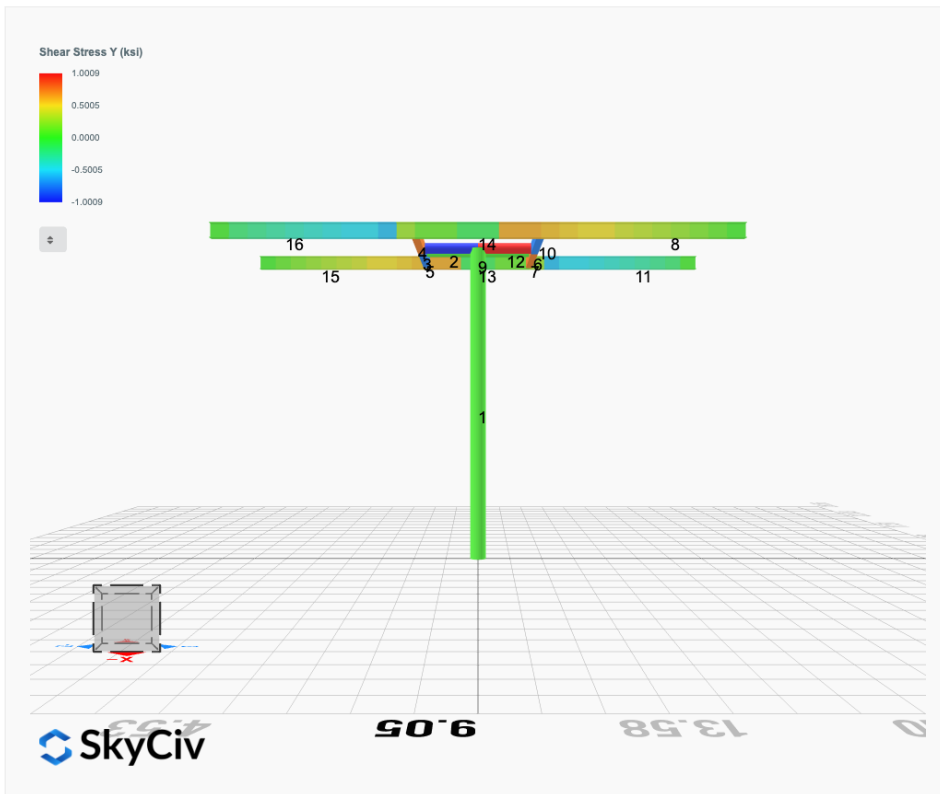
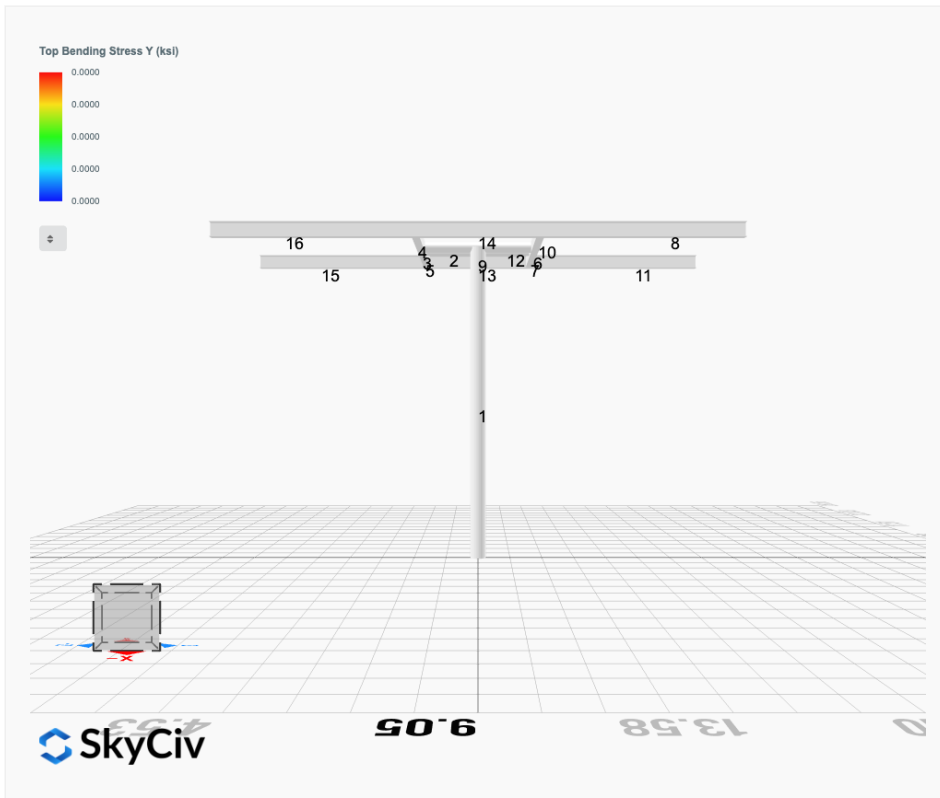
 SkyCiv

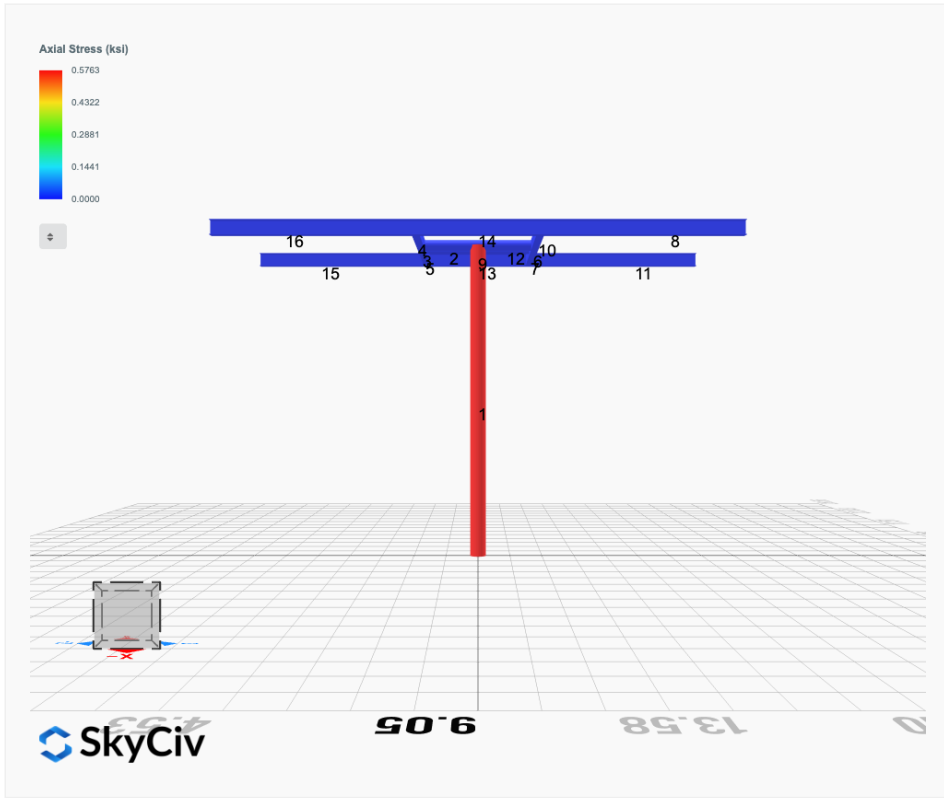


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FEM Results (Envelope Worst Case for each member)







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 | -0.0000 | 2.1041 | 0.0000 | -0.0000 | -0.0000 | 0.0209 |
| ULS: 2. D + L | -0.0000 | 2.1041 | 0.0000 | -0.0000 | -0.0000 | 0.0209 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 5.3205 | 0.0000 | 0.0000 | -0.0000 | 0.0209 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 2.1041 | 0.0000 | -0.0000 | -0.0000 | 0.0209 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 4.5164 | 0.0000 | 0.0000 | -0.0000 | 0.0209 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 2.1041 | 0.0000 | -0.0000 | -0.0000 | 0.0209 |
| ULS: 5b. D + 0.7E | -0.0000 | 2.1041 | 0.0000 | -0.0000 | -0.0000 | 0.0209 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0000 | 4.5164 | 0.0000 | 0.0000 | -0.0000 | 0.0209 |
| ULS: 8. 0.6D + 0.7E | -0.0000 | 1.2625 | 0.0000 | -0.0000 | -0.0000 | 0.0125 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | 0.0000 | 4.2345 | 0.0000 | -0.0000 | 0.0000 | -4.9602 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | 0.0000 | 4.2345 | 0.0000 | -0.0000 | 0.0000 | -4.9602 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | -0.0000 | 4.2345 | 0.0000 | 0.0000 | -0.0000 | 5.0019 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 0.0000 | 0.3998 | 0.0000 | 0.0000 | -0.0000 | -5.1477 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | 0.0000 | 6.1142 | 0.0000 | -0.0000 | 0.0000 | -3.7149 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | 0.0000 | 6.1142 | 0.0000 | -0.0000 | 0.0000 | -3.7149 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | -0.0000 | 6.1142 | 0.0000 | 0.0000 | -0.0000 | 3.7567 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.0000 | 3.2381 | 0.0000 | 0.0000 | -0.0000 | -3.8555 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | 0.0000 | 3.7019 | 0.0000 | -0.0000 | 0.0000 | -3.7149 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | 0.0000 | 3.7019 | 0.0000 | -0.0000 | 0.0000 | -3.7149 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | -0.0000 | 3.7019 | 0.0000 | 0.0000 | -0.0000 | 3.7567 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.0000 | 0.8259 | 0.0000 | 0.0000 | -0.0000 | -3.8555 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | 0.0000 | 3.3929 | 0.0000 | -0.0000 | 0.0000 | -4.9685 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | 0.0000 | 3.3929 | 0.0000 | -0.0000 | 0.0000 | -4.9685 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | -0.0000 | 3.3929 | 0.0000 | 0.0000 | -0.0000 | 4.9936 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 0.0000 | -0.4419 | 0.0000 | 0.0000 | -0.0000 | -5.1560 |

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 | 9.4465 |
| Shear X | -0.0000 |
| Shear Z | 0.0000 |
| Moment X | -0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 8.9754 |

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 | 6.1142 |
| Shear X | -0.0000 |
| Shear Z | 0.0000 |
| Moment X | -0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 5.1560 |

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 | | | | |
| 7 | 6in Pipe Sch 40 | 6.63 | 0.28 | | | | |



| 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 |
| 7 | 6in Pipe Sch 40 | 5.58 | 56.28 | 28.14 | 28.14 | 0.00 | 11.28 | 11.28 |

| | | | | | | |
|----|--------|--------|-------|-------|-------|-------|
| 11 | 120.60 | 34.69 | 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 | 18.13 | 6.45 | 30.09 | 45.74 |
| 14 | 120.60 | 98.23 | 18.13 | 6.45 | 30.09 | 45.74 |
| 15 | 120.60 | 34.69 | 23.36 | 6.45 | 30.09 | 45.74 |
| 16 | 120.60 | 34.69 | 23.36 | 6.45 | 30.09 | 45.74 |

Design Ratio

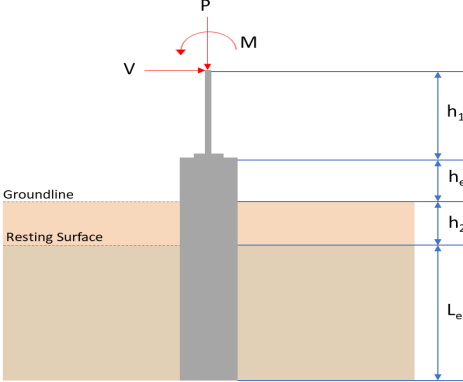
| Member ID | P | M _z | M _y | V _y | V _z | (P,M _z ,M _y) | Worst LC | KL/r | δ | Status |
|-----------|-------|----------------|----------------|----------------|----------------|-------------------------------------|----------|--------------|--------------|--------|
| 1 | 0.114 | 0.212 | 0.000 | 0.000 | 0.000 | 0.253 | #15 | 0.617 | Not Required | Pass |
| 2 | 0.000 | 0.525 | 0.000 | 0.107 | 0.000 | 0.525 | #21 | Not Required | Not Required | Pass |
| 3 | 0.000 | 0.866 | 0.000 | 0.088 | 0.000 | 0.866 | #23 | Not Required | Not Required | Pass |
| 4 | 0.000 | 0.865 | 0.000 | 0.087 | 0.000 | 0.865 | #21 | Not Required | Not Required | Pass |
| 5 | 0.000 | 0.537 | 0.000 | 0.087 | 0.000 | 0.537 | #23 | Not Required | Not Required | Pass |
| 6 | 0.000 | 0.866 | 0.000 | 0.088 | 0.000 | 0.866 | #23 | Not Required | Not Required | Pass |
| 7 | 0.000 | 0.537 | 0.000 | 0.087 | 0.000 | 0.537 | #23 | Not Required | Not Required | Pass |
| 8 | 0.000 | 0.143 | 0.000 | 0.047 | 0.000 | 0.143 | #21 | Not Required | Not Required | Pass |
| 9 | 0.000 | 0.100 | 0.000 | 0.001 | 0.000 | 0.100 | #23 | Not Required | Not Required | Pass |
| 10 | 0.000 | 0.865 | 0.000 | 0.087 | 0.000 | 0.865 | #21 | Not Required | Not Required | Pass |
| 11 | 0.000 | 0.143 | 0.000 | 0.047 | 0.000 | 0.143 | #23 | Not Required | Not Required | Pass |
| 12 | 0.000 | 0.525 | 0.000 | 0.107 | 0.000 | 0.525 | #21 | Not Required | Not Required | Pass |
| 13 | 0.000 | 0.352 | 0.000 | 0.064 | 0.000 | 0.352 | #23 | Not Required | Not Required | Pass |
| 14 | 0.000 | 0.357 | 0.000 | 0.064 | 0.000 | 0.357 | #21 | Not Required | Not Required | Pass |
| 15 | 0.000 | 0.143 | 0.000 | 0.047 | 0.000 | 0.143 | #23 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.143 | 0.000 | 0.047 | 0.000 | 0.143 | #21 | Not Required | 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 |

110

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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 4.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1288 933 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>6.114</td> <td>9.446</td> </tr> <tr> <td>V_x (kip)</td> <td>0.000</td> <td>0.000</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>5.156</td> <td>8.975</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) | 6.114 | 9.446 | V_x (kip) | 0.000 | 0.000 | V_z (kip) | 0.000 | 0.000 | M_x (kipft) | 0.000 | 0.000 | M_z (kipft) | 5.156 | 8.975 | |
| 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) | 6.114 | 9.446 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | 0.000 | 0.000 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.000 | 0.000 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.000 | 0.000 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 5.156 | 8.975 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Required depth to resist lateral loads (ASD) H - Point of application of the lateral load</p> $H = h_1 + h_2 + h_e$ $H = (0 \text{ ft}) + (0 \text{ ft}) + (0 \text{ ft})$ $H = 0 \text{ ft}$ <p>Considering x-direction: H_o - Lateral force per length of pile,</p> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = 0 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(5.156 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.82102 \text{ kipft/ft}$$

L_e - Required depth of embedment in earth,

$$L_e = 2.29 \sqrt[3]{\frac{M_o}{R}}$$

$$L_e = 2.29 \times \sqrt[3]{\frac{(0.82102 \text{ kipft/ft})}{(150 \text{ psf/ft})}}$$

$$L_e = 4.0357 \text{ ft}$$

Considering z-direction:

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

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(4.0357 \text{ ft}), (0 \text{ ft})]$$

$$L_{e,req} = 4.036 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_c - h_2$$

$$L_e = (4.25 \text{ ft}) - (0 \text{ ft}) - (0 \text{ ft})$$

$$L_e = 4.25 \text{ ft}$$

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(4.036 \text{ ft})}{(4.25 \text{ ft})}$$

$$\text{Ratio} = 0.94965$$

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

End-bearing Capacity (ASD)

A - Pile cross-section area

$$A = b D$$

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

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

q - End-bearing pressure

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

$$q = \frac{(6.114 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.38212 \text{ kip/ft}^2$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.38212 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.19106$$

Status: **PASS**
Ratio: **0.190**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

$$L/D = \frac{(4.25 \text{ ft})}{(48 \text{ in})}$$

$$L/D = 1.0625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = 0 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 0.82102 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.82102 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (0.82102 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8333 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (0.82102 \text{ kipft/ft})) + (3 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft}))]^2}{(4.25 \text{ ft})^2 \times [(3 \times (0.82102 \text{ kipft/ft})) + (2 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft}))]}$$

$$p = 0.18182 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (0.82102 \text{ kipft/ft})) + ((0 \text{ kip/ft}) \times (4.25 \text{ ft}))]}{(4.25 \text{ ft})^2}$$

$$s = 0.54545 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(2.8333 \text{ ft})}{2}$$

$$p_a = 0.2125 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.18182 \text{ kip/ft}^2)}{(0.2125 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.85561$$

Status: **PASS**
Ratio: **0.860**

p_s - Allowable lateral soil pressure at depth L_e ,

$$p_s = R L_e$$

$$p_s = (150 \text{ psf/ft}) \times (4.25 \text{ ft})$$

$$p_s = 0.6375 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

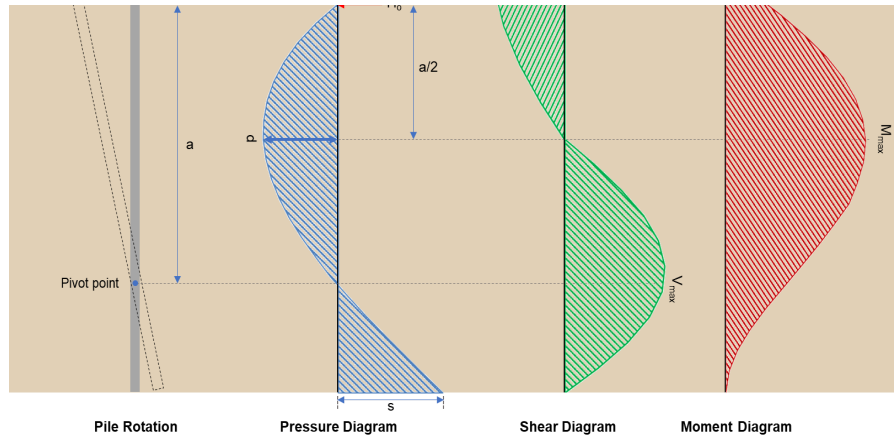
$$\text{Ratio} = \frac{s}{p_s}$$

$$\text{Ratio} = \frac{(0.54545 \text{ kip/ft}^2)}{(0.6375 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.85561$$

Status: **PASS**
Ratio: **0.860**





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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(8.975 \text{ kipft}) + ((0 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 1.4291 \text{ kipft/ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (1.4291 \text{ kipft/ft}) \times (4.25 \text{ ft})) + (3 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft})^2)}{(6 \times (1.4291 \text{ kipft/ft})) + (4 \times (0 \text{ kip/ft}) \times (4.25 \text{ ft}))}$$

$$a = 2.8333 \text{ ft}$$

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

$$V_{max} = 12 \left(\frac{M_o D}{L_c} \right) \left(\frac{a}{L_c} - 1 \right) \left(\frac{a}{L_c} \right)^2$$

$$V_{max} = 12 \times \left(\frac{(1.4291 \text{ kipft/ft}) \times (48 \text{ in})}{(4.25 \text{ ft})} \right) \times \left(\frac{(2.8333 \text{ ft})}{(4.25 \text{ ft})} - 1 \right) \times \left(\frac{(2.8333 \text{ ft})}{(4.25 \text{ ft})} \right)^2$$

$$V_{max} = 2.3912 \text{ kip}$$

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

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

$$M_{max} = ((1.4291 \text{ kipft/ft}) \times (48 \text{ in})) \times \left[1 - \left(4 \times \frac{(2.8333 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^3 + \left(3 \times \frac{(2.8333 \text{ ft})}{2 \times (4.25 \text{ ft})} \right)^4 \right]$$

$$M_{max} = 5.0814 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Longitudinal reinforcement:

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

Table 22.4.2.1

22.4.2.2, 10.6.1.1

| | | |
|-----------------|--|--|
| | $A_{st,required} = Min \left[\frac{\phi \alpha \sqrt{f_{ck}} J_{ck}^{1/4}}{f_{yk} - (0.85 f_{ck})}, (0.08 A_g) \right]$ $A_{st,required} = Min \left[\frac{\frac{(9.446 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (3 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$ $A_{st,required} = -101.95 \text{ in}^2$ <p><i>A_{min}</i> - Governing minimum reinforcement area,</p> $A_{min} = Max [A_{st,required}, (0.0018 A_g)]$ $A_{min} = Max [(-101.95 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$ $A_{min} = 4.1472 \text{ in}^2$ <p><i>n_{rebar}</i> - Required number of reinforcement,</p> $n_{rebar} = \frac{A_{min}}{A_{rebar}}$ $n_{rebar} = \frac{(4.1472 \text{ in}^2)}{(0.3068 \text{ in}^2)}$ $n_{rebar} = 14$ <p><i>A_{st}</i> - Actual total reinforcement area,</p> $A_{st} = n_{rebar} \frac{\pi d_{bar}^2}{4}$ $A_{st} = (14) \times \frac{\pi \times (0.625 \text{ in})^2}{4}$ $A_{st} = 4.2951 \text{ in}^2$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{A_{min}}{A_{st}}$ $Ratio = \frac{(4.1472 \text{ in}^2)}{(4.2951 \text{ in}^2)}$ $Ratio = 0.96556$ <p>25.2.3 <i>s_{rebar}</i> - Minimum spacing of reinforcement,</p> $s_{rebar} = Max [1.5, (1.5 d_{bar})]$ $s_{rebar} = Max [1.5, (1.5 \times (0.625 \text{ in}))]$ $s_{rebar} = 1.5 \text{ in}$ <p>Ties:</p> <p>25.7.2.2 Since longitudinal reinforcement is ≤ No. 10Ø: Use #3(0.375 in)</p> <p>25.7.2.1 <i>s_{ties}</i> - Maximum spacing of ties,</p> $s_{ties} = Min [(16 d_{bar}), (48 d_{ties}), Min (D, b)]$ $s_{ties} = Min [(16 \times (0.625 \text{ in})), (48 \times (0.375 \text{ in})), Min ((48 \text{ in}), (48 \text{ in}))]$ $s_{ties} = 10 \text{ in}$ <p>Summary:</p> <p>Main reinforcement: 14 - #5 (0.625 in) Ties: #3(0.375 in) - 10 in</p> | <p>Status: PASS Ratio: 0.970</p> |
| <p>22.4.2.2</p> | <p>Axial Compression Strength (ACI 318-19, LRFD)</p> <p><i>φP_N</i> - Allowable axial compressive strength</p> $\phi P_N = \phi 0.80 [(0.85 f'_{ck} [A_g - A_{st}]) + (f_{yk} A_{st})]$ $\phi P_N = (0.65) \times 0.80 \times [(0.85 \times (3 \text{ ksi}) \times (2304 \text{ in}^2) - (4.2951 \text{ in}^2))] + ((60 \text{ ksi}) \times (4.2951 \text{ in}^2))]$ $\phi P_N = 3183.4 \text{ kip}$ <p><i>Ratio</i> - Capacity</p> $Ratio = \frac{P}{\phi P_N}$ | |

$$Ratio = \frac{(9.446 \text{ kip})}{(3183.4 \text{ kip})}$$

$$Ratio = 0.0029673$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

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

$$d = 0.80 D$$

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

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

22.5.5.1.3 λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

22.5.5.1.1(a) The following variables were converted to be consistent with empirical formula $f'_{ck} = 3 \text{ ksi} \rightarrow 3000 \text{ psi}$, $P = 9.446 \text{ kip} \rightarrow 9446 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(3000 \text{ psi})} + \frac{(9446 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 131.05 \text{ kip}$$

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min} [(324.49 \text{ kip}), (131.05 \text{ kip}), (406.27 \text{ kip})]$$

$$V_c = 131.05 \text{ kip}$$

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

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

$$V_{s,a} = 8 \times \sqrt{(3000 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{s,a} = 807.65 \text{ kip}$$

A_v - Ties rebar area,

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

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

| | | |
|---|---|---|
| <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> <p>22.5.1.1 ϕV_n - Allowable shear strength</p> | $A_v = \frac{A_v}{4}$ $A_v = 0.11045 \text{ in}^2$ $V_{s,b} = \frac{2 A_v f_{ywk} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(807.65 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((131.05 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 118.27 \text{ kip}$ | |
| <p>14.5.2.1b $\phi M_{n,2}$</p> | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{(3 \text{ ksi})} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 273.423 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 f'_c S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (3 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2545.9 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(273.42 \text{ kipft}), (2545.9 \text{ kipft})]$ $\phi M_n = 273.42 \text{ kipft}$ <p>Considering x-direction:</p> <p>$M_{max} = 5.0814 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(5.0814 \text{ kipft})}{(273.42 \text{ kipft})}$ $\text{Ratio} = 0.018584$ | <p>Status: PASS Ratio: 0.020</p> |

