# Your Project Calculations

Project Name: Johnazreian A CU

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Johnazreian%20A%20CU&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/8_2023

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=7FhwJQwmgzYlBj1Ii5ZuQenJA1w7HPfrPKe38i8X5nyg0FuZJKZBDMLAPRi3siic

## Array Specification

| | |
|---|---|
| **Product:** | Beam |
| **Unique ID:** | 1P-0-6TOP-SD-24-L-4Hx2W-6KIH |
| **Duty Classification:** | SD |
| **Module Width:** | 40.00 in |
| **Module Length:** | 72.00in |
| **Number of Rows:** | 4 |
| **Number of Columns:** | 2 |
| **Total Number of Modules:** | 8 |
| **Desired Tilt Angle:** | 45 |
| **Front Edge Clearance:** | 5 |
| **Total Array Height at Tilt:** | 14.49 ft |
| **Total Frame Length:** | 11.50 ft |
| **Frame Weight:** | 544 lbs |
| **Array Dimensions N/S:** | 13.50 ft |
| **Array Dimensions E/W:** | 12.17 ft |
| **Rail Length:** | 162.00 in |
| **Rail Spacing:** | 3.00 ft |
| **Rail Check:** | Not Checked |

## Support Specifications

| | |
|---|---|
| **Pole Size:** | 6in Pipe Sch 40 |
| **Pole Length above Grade:** | 9.77 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: 5.75 ft |
| **Foundation Volume:** | 3.407 $y^3$ |
| **Foundation Result:** | PASSED |
| **Mount Twist:** | 0.000003 kip |

## Site Info

| | |
|---|---|
| **Risk Category:** | I |
| **Exposure:** | C |
| **Soil Classification:** | sand |
| **Site Location:** | 2790 Skyline Dr, New Meadows, ID 83654, USA |
| **Wind Speed:** | 95 mph |
| **Snow Load:** | 85.98 psf |
| **Design Uplift Pressure:** | Multiple pressures |
| **Design Downforce Pressure:** | Multiple pressures |
| **Design Snow Pressure:** | 0.023637 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


```
{"wind_speed_override":null,"snow_load_override":null,"direct_snow_load":false,"product_type":"Beam","project_id":"Johnazreian A
CU","site_address":"2790 Skyline Dr, New Meadows, ID 83654,
USA","module_width":40,"module_length":72,"number_rows":"4","number_columns":2,"pole_mount_section":"4_40","core_pipe_width":65,
"core_pipe_section":"2_40","adjuster_section":"2_40","core_beam_height":65,"core_beam_section":"HSS3x2x1/8","main_pipe_section":
"2_12GA","pole_spacing":"15","tilt_angle":45,"ground_clearance":5,"risk_category":"I","exposure_category":"C","frame_duty_overri
de":"auto","pole_override":"auto","soil_type":"sand","customer_foundation_override":"48_Square","foundation_type":"Square","foun
dation_size":48,"check_rails":false}
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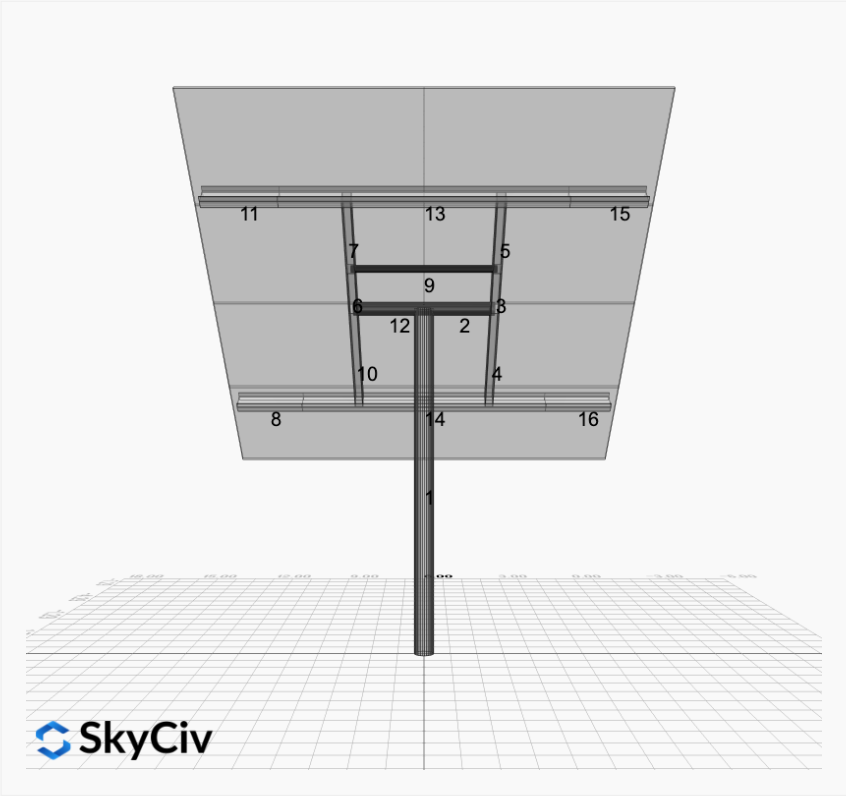
**Design Notes:**

- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Design and Sizing is approximate only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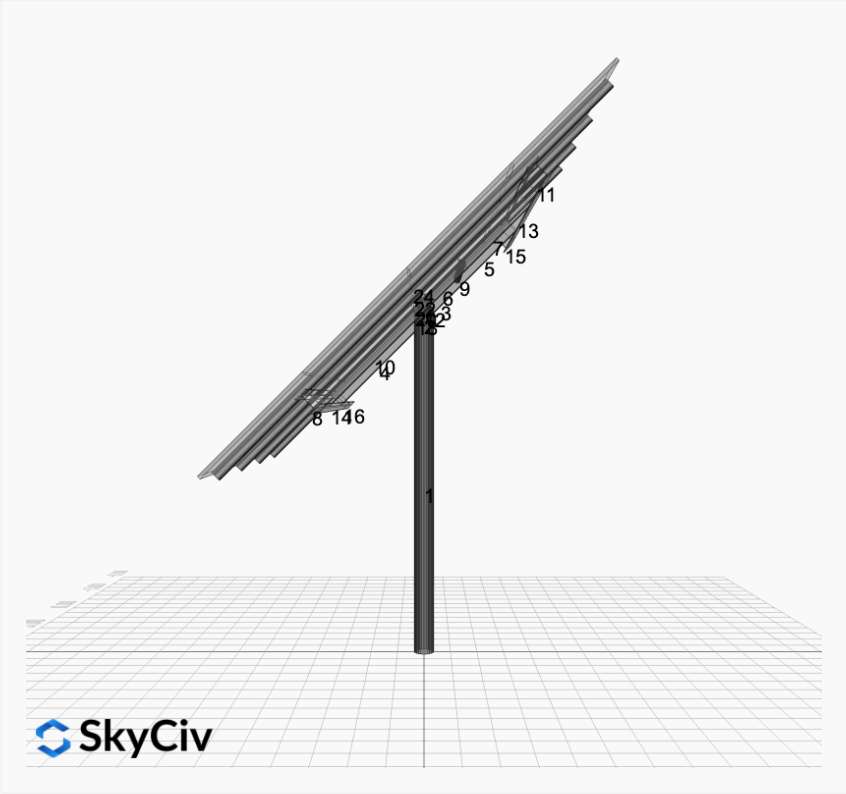
13
15
7
5
9
6
3
12
2
10
4
8
14
16
1
SkyCiv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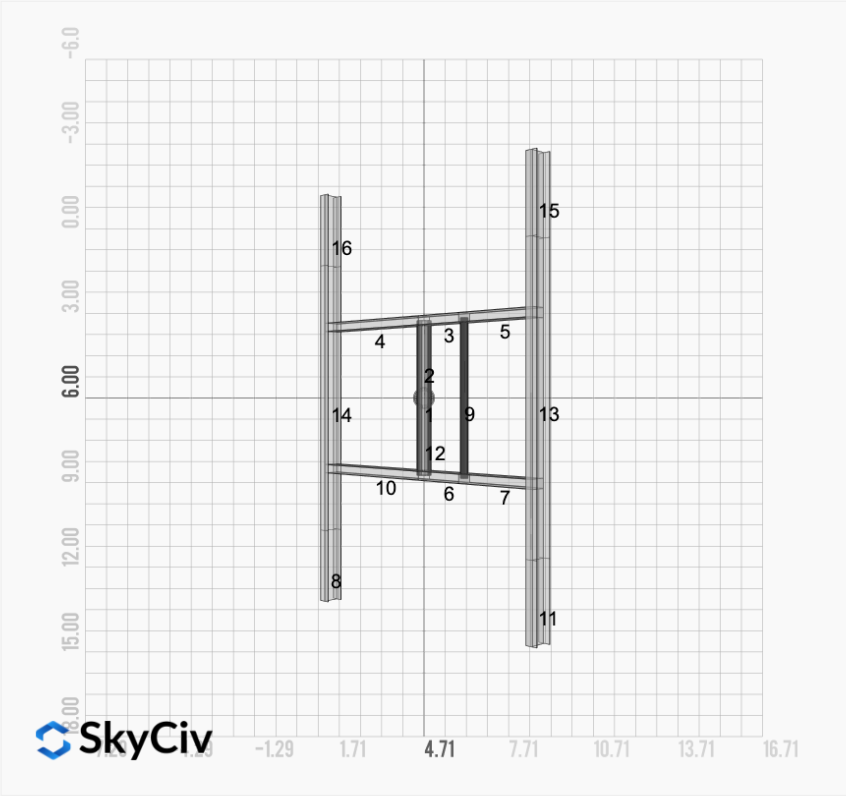
13
7
15
5
9
10
4
8
14
6
1
SkyCiv


-6.0
-3.00
0.00
3.00
6.00
9.00
12.00
15.00
18.00
-1.29
1.71
4.71
7.71
10.71
13.71
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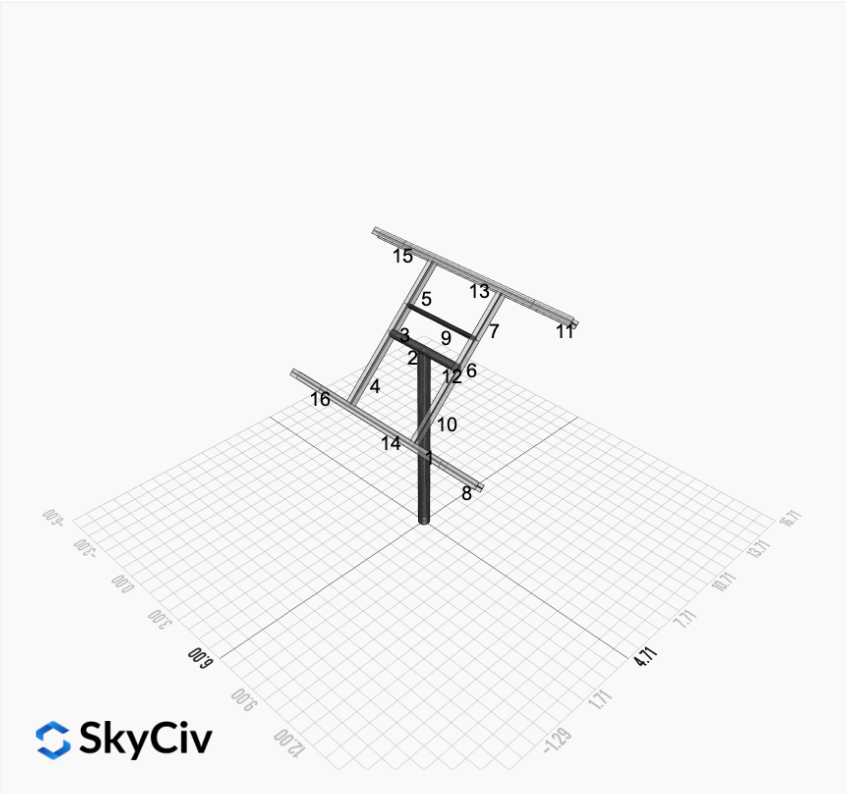
10
11
12
13
14
15
16
SkyCiv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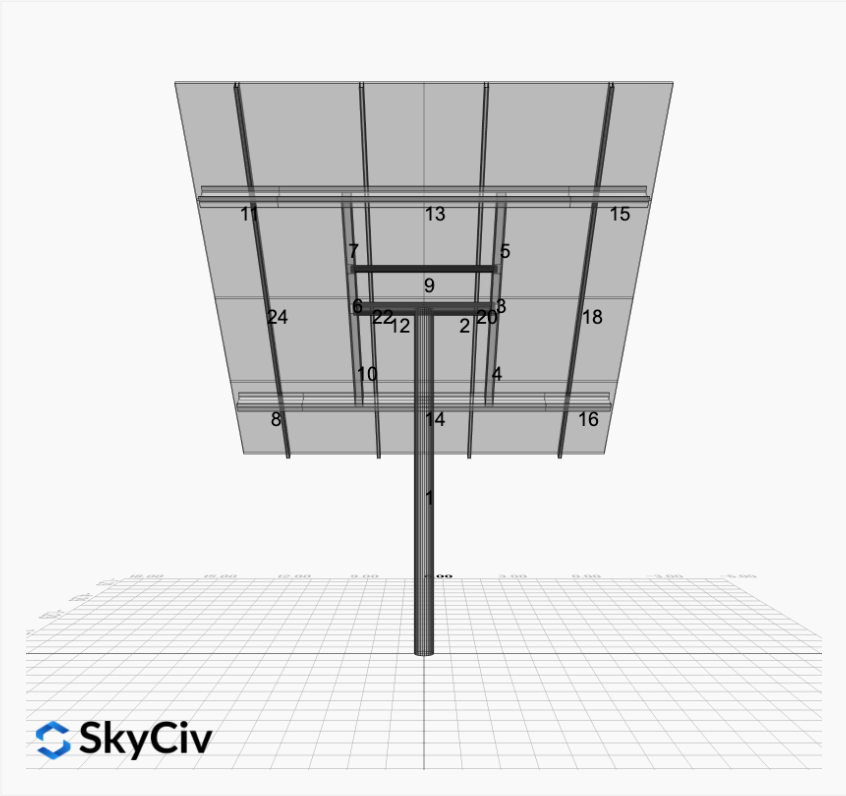
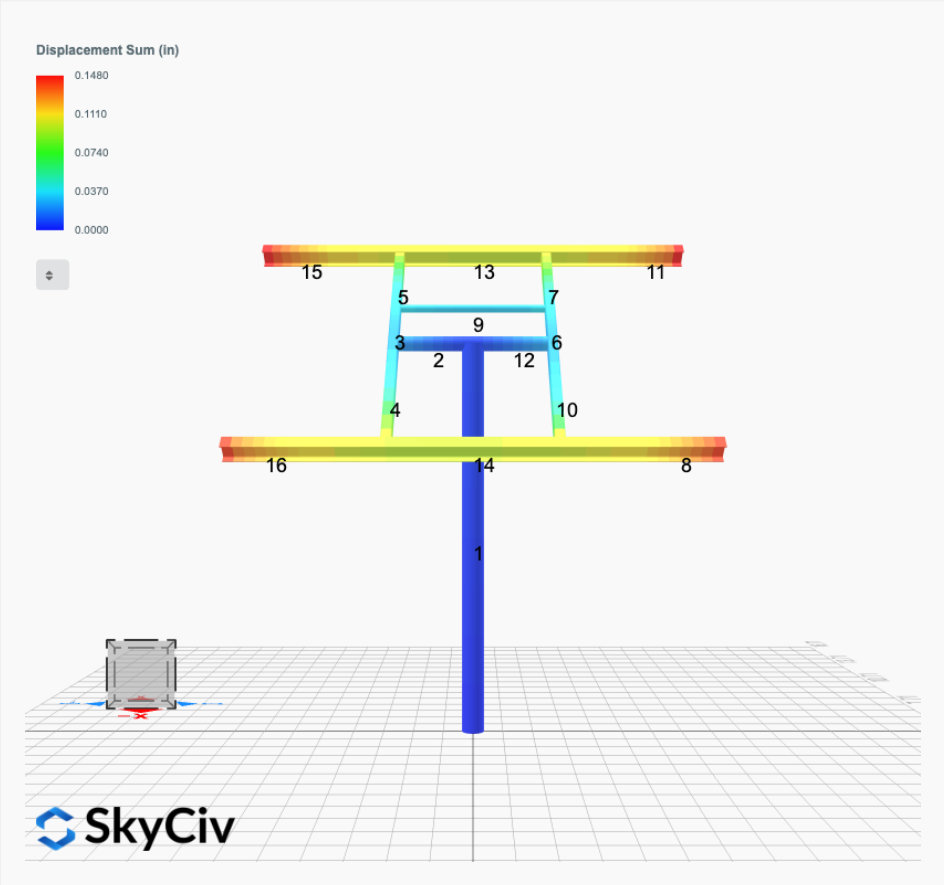
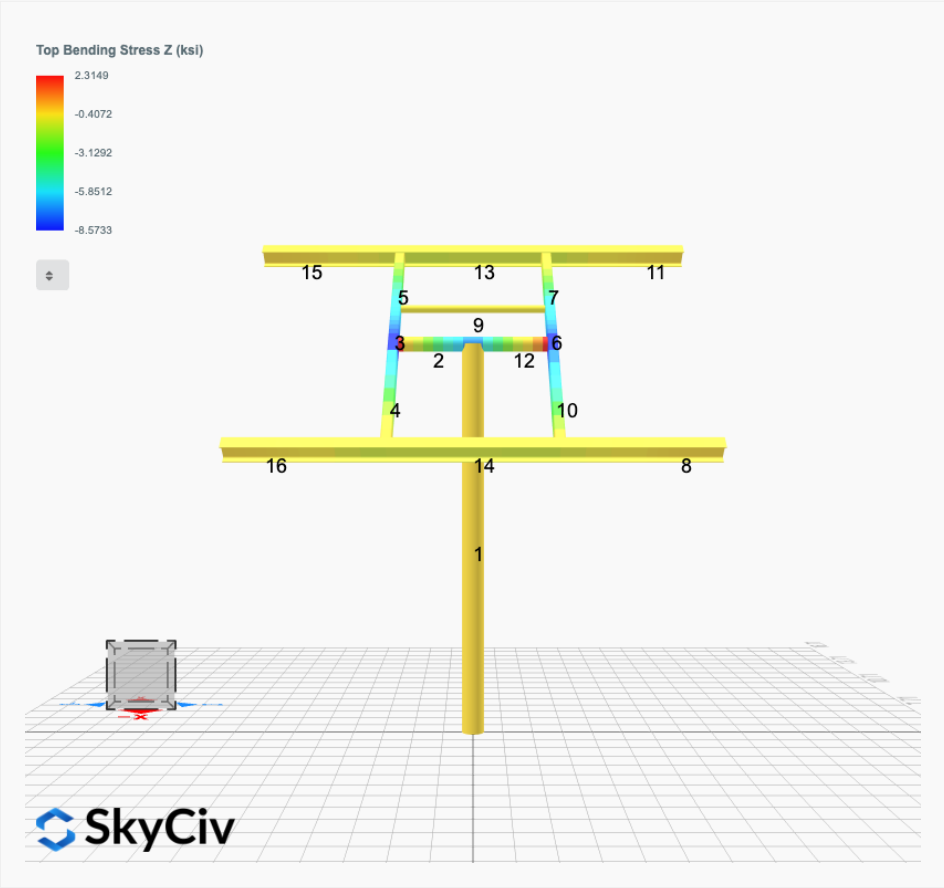
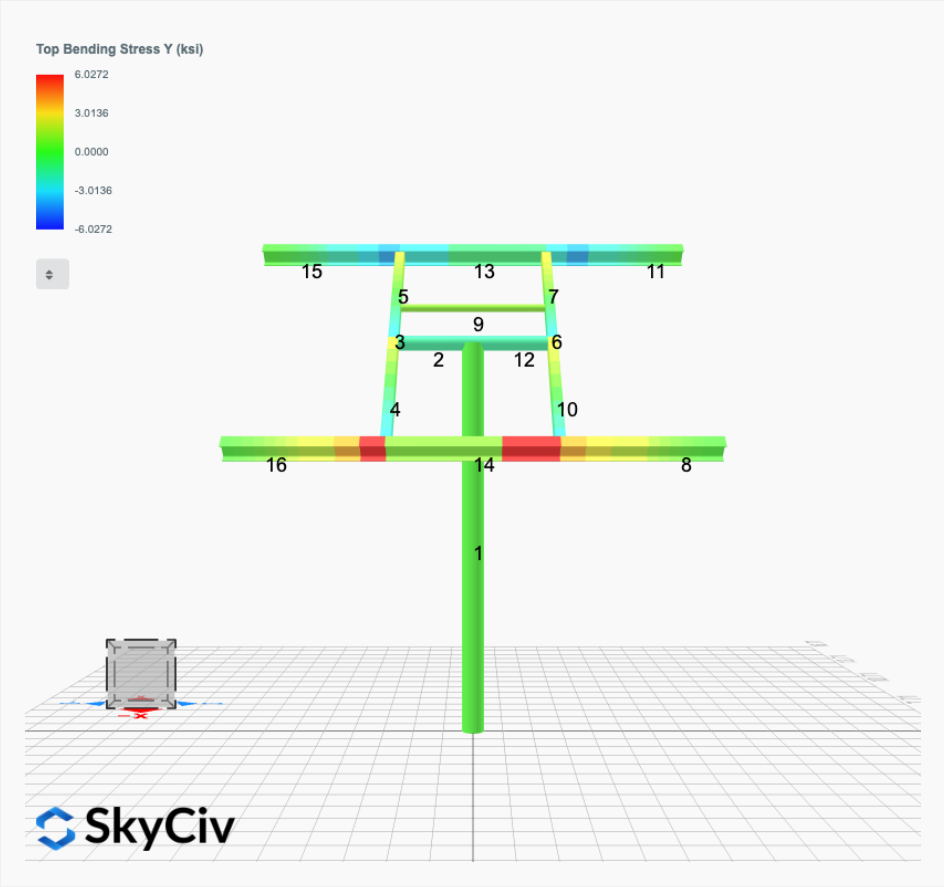
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
-1.29
1.71
4.71
7.71
10.71
13.71
16.71
-3.00
0.00
3.00
6.00
9.00
12.00
SkyCiv

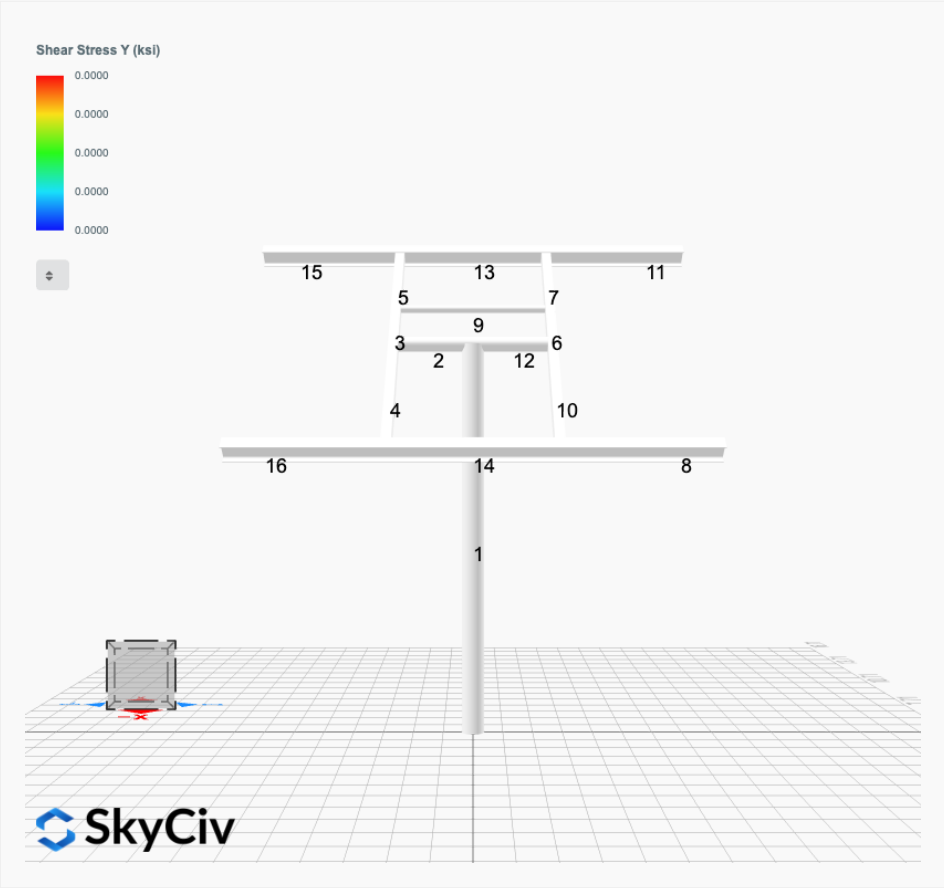

1
13
15
7
5
9
6
22
12
2
20
8
24
18
10
4
8
14
16
1
SkyCiv

## FEM Results (Envelope Worst Case for each member)




Top Bending Stress Z (ksi)
2.3149
-0.4072
-3.1292
-5.8512
-8.5733
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Top Bending Stress Y (ksi)
6.0272
3.0136
0.0000
-3.0136
-6.0272
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Shear Stress Y (ksi)
0.0000
0.0000
0.0000
0.0000
0.0000
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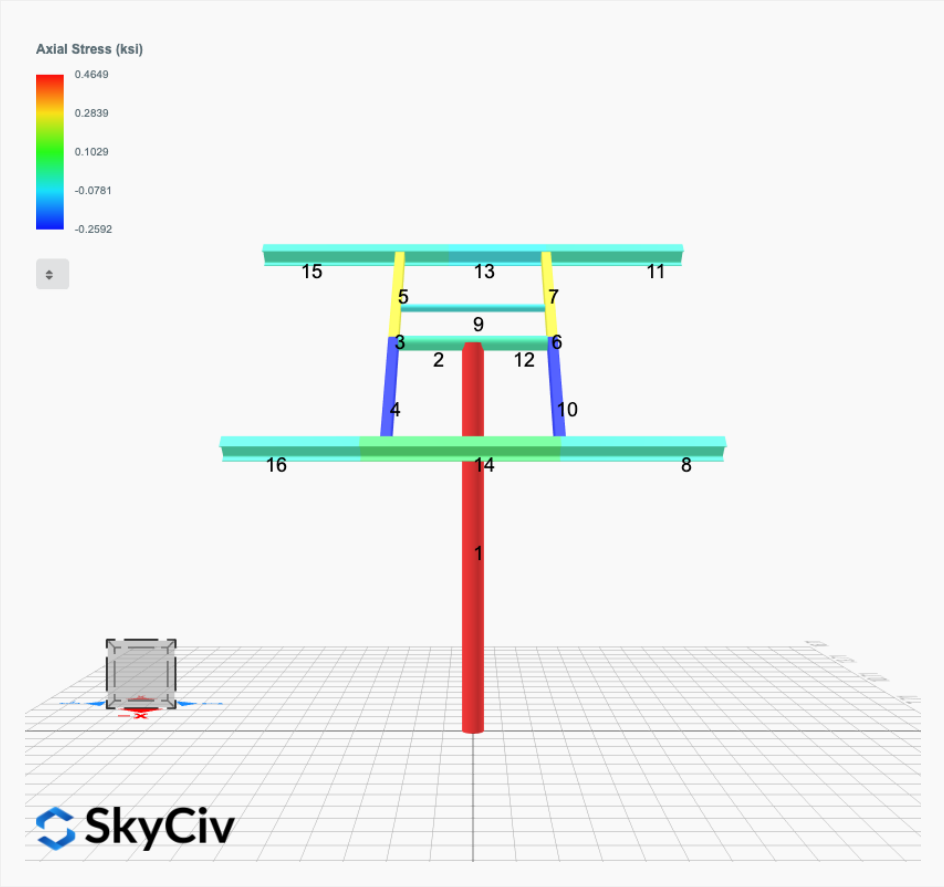
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv


Axial Stress (ksi)
0.4649
0.2839
0.1029
-0.0781
-0.2592
15
13
11
5
7
9
3
6
2
12
4
10
16
14
8
1
SkyCiv

## 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 | 1.3205 | -0.0000 | 0.0000 | -0.0000 | 0.0159 |
| ULS: 2. D + L | 0.0000 | 1.3205 | -0.0000 | 0.0000 | -0.0000 | 0.0159 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 3.9153 | -0.0000 | 0.0000 | -0.0000 | 0.0284 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 1.3205 | -0.0000 | 0.0000 | -0.0000 | 0.0159 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 3.2666 | -0.0000 | 0.0000 | -0.0000 | 0.0253 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 1.3205 | -0.0000 | 0.0000 | -0.0000 | 0.0159 |
| ULS: 5b. D + 0.7E | 0.0000 | 1.3205 | -0.0000 | 0.0000 | -0.0000 | 0.0159 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0000 | 3.2666 | -0.0000 | 0.0000 | -0.0000 | 0.0253 |
| ULS: 8. 0.6D + 0.7E | 0.0000 | 0.7923 | -0.0000 | 0.0000 | -0.0000 | 0.0096 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -1.9866 | 3.3071 | -0.0000 | 0.0000 | -0.0000 | 20.4922 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -1.9866 | 3.3071 | -0.0000 | 0.0000 | -0.0000 | 20.4922 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 1.4371 | -0.1166 | -0.0000 | 0.0000 | -0.0000 | -13.4037 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 1.2681 | 0.0525 | -0.0000 | 0.0000 | -0.0000 | -15.7554 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.4900 | 4.7566 | -0.0000 | 0.0000 | -0.0000 | 15.3825 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.4900 | 4.7566 | -0.0000 | 0.0000 | -0.0000 | 15.3825 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0778 | 2.1888 | -0.0000 | 0.0000 | -0.0000 | -10.0395 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.9510 | 2.3156 | -0.0000 | 0.0000 | -0.0000 | -11.8032 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.4900 | 2.8105 | -0.0000 | 0.0000 | -0.0000 | 15.3731 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.4900 | 2.8105 | -0.0000 | 0.0000 | -0.0000 | 15.3731 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0778 | 0.2427 | -0.0000 | 0.0000 | -0.0000 | -10.0488 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.9510 | 0.3695 | -0.0000 | 0.0000 | -0.0000 | -11.8125 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -1.9866 | 2.7789 | -0.0000 | 0.0000 | -0.0000 | 20.4858 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -1.9866 | 2.7789 | -0.0000 | 0.0000 | -0.0000 | 20.4858 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 1.4371 | -0.6448 | -0.0000 | 0.0000 | -0.0000 | -13.4101 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 1.2681 | -0.4757 | -0.0000 | 0.0000 | -0.0000 | -15.7617 |

### Worst Case Reactions LRFD

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

| Result | Value (kip, kip-ft) |
|---|---|
| Axial | 7.3918 |
| Shear X | -3.3110 |
| Shear Z | -0.0000 |
| Moment X | -0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 34.7792 |

### 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 | 4.7566 |
| Shear X | -1.9866 |
| Shear Z | -0.0000 |
| Moment X | 0.0000 |
| Moment Y (Twist) | 0.0000 |
| Moment Z | 20.4922 |

## Project Details

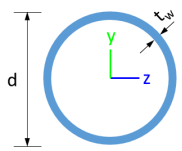
| | |
|---|---|
| Design Code: | AISC 360-16 LRFD |
| Provision: | LRFD |
| Country: | United States |
| User Name: | sales@mtsolar.us |
| Unit System: | imperial |

## Design Input Information

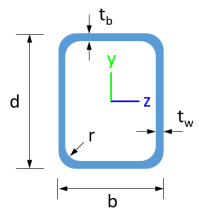
| Design Factors | | | |
|---|---|---|---|
| $\Phi_t$ | $\Phi_c$ | $\Phi_b$ | $\Phi_v$ |
| 0.9 | 0.9 | 0.9 | 0.9 |

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

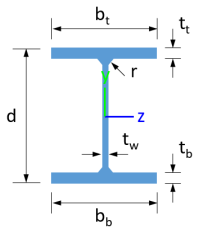
**Section Dimensions**




| 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 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 15 | HSS5x3x1/8 | 1.77 | 6.02 | 2.75 | 6.03 | 152.35 | 2.07 | 2.93 |
| 18 | W6x9 | 2.68 | 0.04 | 2.20 | 16.40 | 17.70 | 1.72 | 6.23 |

| Member Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Member ID | Section ID | $K_zL$ (ft) | $K_yL$ (ft) | $L_b$ (ft) | $C_b$ | LST | LSC | LD |
| 1 | 7 | 20.52 | 20.52 | 9.77 | - | 300 | 200 | 1 |
| 2 | 4 | 1.30 | 1.30 | 2.00 | - | 300 | 200 | 1 |
| 3 | 15 | 0.92 | 0.92 | 1.42 | 1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18 | 300 | 200 | 1 |
| 4 | 15 | 2.44 | 2.44 | 3.75 | 1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.79,1.67,1.67,1.66,1.65 | 300 | 200 | 1 |
| 5 | 15 | 1.52 | 1.52 | 2.33 | 1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66 | 300 | 200 | 1 |
| 6 | 15 | 0.92 | 0.92 | 1.42 | 1.19,1.18,1.19,1.18,1.19,1.19,1.18,1.18,1.17,1.17,1.18,1.18,1.17,1.18,1.18,1.18,1.18,1.18,1.18,1.18,1.16,1.17,1.18,1.18,1.18,1.18 | 300 | 200 | 1 |
| 7 | 15 | 1.52 | 1.52 | 2.33 | 1.68,1.67,1.68,1.67,1.68,1.68,1.67,1.67,1.66,1.66,1.67,1.67,1.66,1.66,1.67,1.67,1.67,1.67,1.67,1.67,1.65,1.66,1.67,1.67,1.66,1.66 | 300 | 200 | 1 |
| 8 | 18 | 4.20 | 4.20 | 2.00 | 2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33 | 300 | 200 | 1 |
| 9 | 1 | 2.60 | 2.60 | 4.00 | - | 300 | 200 | 1 |
| 10 | 15 | 2.44 | 2.44 | 3.75 | 1.69,1.68,1.69,1.67,1.68,1.69,1.67,1.67,1.66,1.72,1.67,1.67,1.66,1.63,1.67,1.67,1.68,1.67,1.67,1.67,1.65,1.79,1.67,1.67,1.66,1.65 | 300 | 200 | 1 |
| 11 | 18 | 4.20 | 4.20 | 2.00 | 2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33 | 300 | 200 | 1 |
| 12 | 4 | 1.30 | 1.30 | 2.00 | - | 300 | 200 | 1 |
| 13 | 18 | 4.88 | 4.00 | 7.50 | 1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.08,1.09,1.09,1.09,1.10,1.09,1.09,1.09,1.10 | 300 | 200 | 1 |
| 14 | 18 | 4.88 | 4.00 | 7.50 | 1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,1.09,2.99,1.09,1.09,1.09,1.06,1.09,1.09,1.09,1.09,1.09,1.09,3.50,1.09,1.09,1.09,1.06 | 300 | 200 | 1 |
| 15 | 18 | 4.20 | 4.20 | 2.00 | 2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33 | 300 | 200 | 1 |
| 16 | 18 | 4.20 | 4.20 | 2.00 | 2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33,2.33 | 300 | 200 | 1 |

## Member Design Capacity

| Member ID | $\Phi_t P_n$ (kip) | $\Phi_c P_n$ (kip) | $\Phi_b M_{zn}$ (k-ft) | $\Phi_b M_{yn}$ (k-ft) | $\Phi_v V_{yn}$ (kip) | $\Phi_v V_{zn}$ (kip) |
|---|---|---|---|---|---|---|
| 1 | 251.16 | 104.23 | 42.30 | 42.30 | 75.35 | 75.35 |
| 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 | 96.18 | 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 | 96.18 | 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 | 19.01 | 6.45 | 30.09 | 45.74 |
| 14 | 120.60 | 98.23 | 18.66 | 6.45 | 30.09 | 45.74 |
| 15 | 120.60 | 96.18 | 23.36 | 6.45 | 30.09 | 45.74 |
| 16 | 120.60 | 96.18 | 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 | 0.071 | 0.822 | 0.000 | 0.044 | 0.000 | 0.852 | #13 | 0.548 | Not Required | Pass |
| 2 | 0.001 | 0.350 | 0.204 | 0.084 | 0.039 | 0.511 | #13 | 0.052 | Not Required | Pass |
| 3 | 0.013 | 0.585 | 0.166 | 0.059 | 0.029 | 0.718 | #21 | 0.044 | Not Required | Pass |
| 4 | 0.012 | 0.531 | 0.160 | 0.053 | 0.023 | 0.668 | #21 | 0.078 | Not Required | Pass |
| 5 | 0.013 | 0.363 | 0.157 | 0.058 | 0.021 | 0.375 | #13 | 0.073 | Not Required | Pass |
| 6 | 0.013 | 0.585 | 0.166 | 0.059 | 0.029 | 0.718 | #21 | 0.044 | Not Required | Pass |
| 7 | 0.013 | 0.363 | 0.157 | 0.058 | 0.021 | 0.375 | #13 | 0.073 | Not Required | Pass |
| 8 | 0.000 | 0.023 | 0.051 | 0.018 | 0.007 | 0.073 | #21 | Not Required | Not Required | Pass |
| 9 | 0.003 | 0.038 | 0.043 | 0.001 | 0.000 | 0.080 | #13 | 0.132 | Not Required | Pass |
| 10 | 0.012 | 0.531 | 0.160 | 0.053 | 0.023 | 0.668 | #21 | 0.078 | Not Required | Pass |
| 11 | 0.000 | 0.025 | 0.051 | 0.019 | 0.007 | 0.074 | #21 | Not Required | Not Required | Pass |
| 12 | 0.001 | 0.350 | 0.204 | 0.084 | 0.039 | 0.511 | #13 | 0.052 | Not Required | Pass |
| 13 | 0.003 | 0.120 | 0.179 | 0.036 | 0.013 | 0.279 | #21 | 0.177 | Not Required | Pass |
| 14 | 0.004 | 0.114 | 0.179 | 0.033 | 0.013 | 0.274 | #21 | 0.177 | Not Required | Pass |
| 15 | 0.000 | 0.025 | 0.051 | 0.019 | 0.007 | 0.074 | #21 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.023 | 0.051 | 0.018 | 0.007 | 0.073 | #21 | Not Required | Not Required | Pass |

## Definitions

| | |
|---|---|
| $\Phi_t$ | Safety factor for tensile |
| $\Phi_c$ | Safety factor for compression |
| $\Phi_b$ | Safety factor for flexure |
| $\Phi_v$ | 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/comression) |
| $M_n$ | 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 |

| | |
|---|---|
| NS | Capacity is not provided |

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

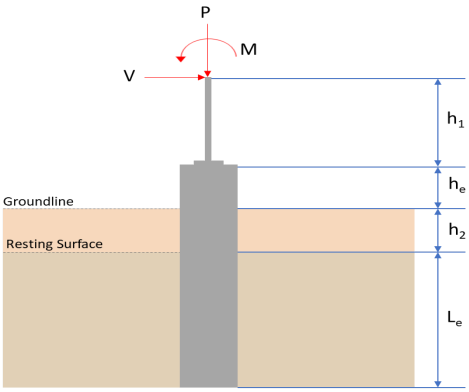
## SkyCiv Foundation Design

Pile Foundation

**Design Information :**

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

### Pile Input




**Geometry**

Pile shape: rectangular
$b = 48\ \text{in}$ - Pile width
$D = 48\ \text{in}$ - Pile depth
$L = 5.75\ \text{ft}$ - Total pile length
$h_1 = 0\ \text{ft}$ - Lateral load height from the top of the pile,
$h_2 = 0\ \text{ft}$ - Depth to resisting surface
$h_e = 0\ \text{ft}$ - Length of pile above the ground

**Tabulation of Soil Parameters**

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

**Tabulation of Loads**

| Load Component | ASD | LRFD |
|---|---|---|
| $P$ (kip) | 4.757 | 7.392 |
| $V_x$ (kip) | -1.987 | -3.311 |
| $V_z$ (kip) | 0.000 | 0.000 |
| $M_x$ (kipft) | 0.000 | 0.000 |
| $M_z$ (kipft) | 20.492 | 34.779 |

**Material Properties**

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

### Required depth to resist lateral loads (ASD)

$H$ - Point of application of the lateral load

$$H = h_1 + h_2 + h_e$$

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

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

**Considering x-direction:**

$H_o$ - Lateral force per length of pile,

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

$$H_o = \frac{(-1.987\ \text{kip})}{1.57 \times (48\ \text{in})}$$

$$H_o = -0.3164\ \text{kip/ft}$$

$M_o$ - Moment per length of pile,

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

$$M_o = \frac{(20.492\ \text{kipft}) + ((-1.987\ \text{kip}) \times (0\ \text{ft}))}{1.57 \times (48\ \text{in})}$$

$$M_o = 3.2631\ \text{kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:
$L_{e,x} = 5.41\ \text{ft}$ - Required depth in x-direction,

**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,

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

$$L_{e,req} = MAX[(5.41\ \text{ft}), (0\ \text{ft})]$$

$$L_{e,req} = 5.41\ \text{ft}$$

$L_e$ - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

$$L_e = (5.75\ \text{ft}) - (0\ \text{ft}) - (0\ \text{ft})$$

$$L_e = 5.75\ \text{ft}$$

$Ratio$ - Embedded depth

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

$$Ratio = \frac{(5.41\ \text{ft})}{(5.75\ \text{ft})}$$

$$Ratio = 0.94087$$

Status: **PASS**
Ratio: **0.940**

**End-bearing Capacity (ASD)**

$A$ - Pile cross-section area

$$A = b\,D$$

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

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

$q$ - End-bearing pressure

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

$$q = \frac{(4.757\ \text{kip})}{(16\ \text{ft}^2)}$$

$$q = 0.29731\ \text{kip/ft}^2$$

**Check bearing capacity ratio:**

$Ratio$ - Capacity

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

$$Ratio = \frac{(0.29731\ \text{kip/ft}^2)}{(2000\ \text{psf})}$$

$$Ratio = 0.14866$$

Status: **PASS**
Ratio: **0.150**

*Czerniak*

**Lateral Soil Pressure (ASD):**

$L/D$ - Length to least lateral dimension ratio,

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

$$L/D = \frac{(5.75\ \text{ft})}{(48\ \text{in})}$$

$$L/D = 1.4375$$

Since L/D $\leq$ 10,

Pile is short.

**Considering x-direction:**

$H_o = -0.3164\ \text{kip/ft}$ - Lateral force per length of pile,
$M_o = 3.2631\ \text{kipft/ft}$ - Overturning moment per length of pile,
$a$ - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.2631\ \text{kipft/ft}) \times (5.75\ \text{ft})) + \left(3 \times (-0.3164\ \text{kip/ft}) \times (5.75\ \text{ft})^2\right)}{(6 \times (3.2631\ \text{kipft/ft})) + (4 \times (-0.3164\ \text{kip/ft}) \times (5.75\ \text{ft}))}$$

$$a = 3.9632\ \text{ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.2631\ \text{kipft/ft})) + (3 \times (-0.3164\ \text{kip/ft}) \times (5.75\ \text{ft}))]^{\ 2}}{(5.75\ \text{ft})^2 \times [(3 \times (3.2631\ \text{kipft/ft})) + (2 \times (-0.3164\ \text{kip/ft}) \times (5.75\ \text{ft}))]}$$

$$p = 0.21271\ \text{kip/ft}^2$$

$s$ - Earth pressure against the pile at distance $L_e$,

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

$$s = \frac{6 \times [(2 \times (3.2631\ \text{kipft/ft})) + ((-0.3164\ \text{kip/ft}) \times (5.75\ \text{ft}))]}{(5.75\ \text{ft})^2}$$

$$s = 0.85417\ \text{kip/ft}^2$$

**Check lateral soil pressure capacity:**

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

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

$$p_a = (150\ \text{psf/ft}) \times \frac{(3.9632\ \text{ft})}{2}$$

$$p_a = 0.29724\ \text{kip/ft}^2$$

$Ratio$ - Lateral soil capacity

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

$$Ratio = \frac{(0.21271\ \text{kip/ft}^2)}{(0.29724\ \text{kip/ft}^2)}$$

$$Ratio = 0.71562$$

Status: **PASS**
Ratio: **0.720**

$p_s$ - Allowable lateral soil pressure at depth $L_e$,

$$p_s = R\ L_e$$

$$p_s = (150\ \text{psf/ft}) \times (5.75\ \text{ft})$$

$$p_s = 0.8625\ \text{kip/ft}^2$$

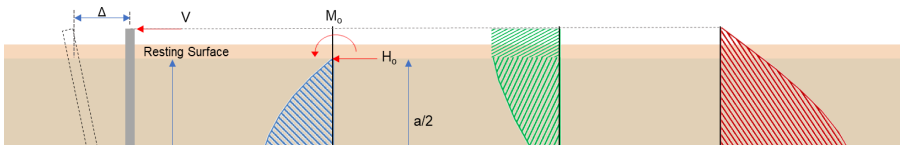
$Ratio$ - Lateral soil capacity

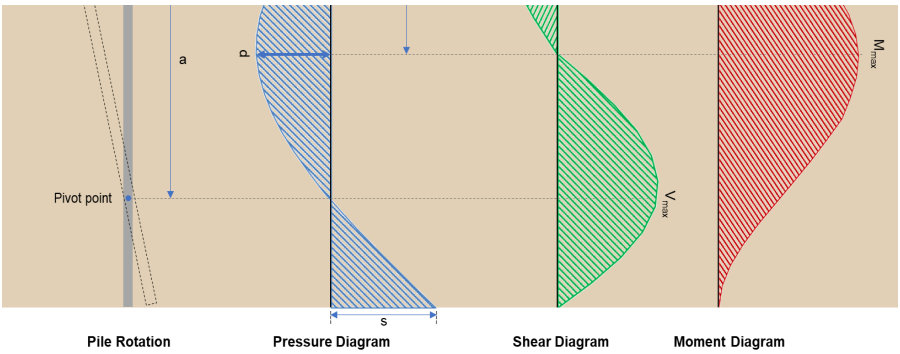
$$Ratio = \frac{s}{p_s}$$

$$Ratio = \frac{(0.85417\ \text{kip/ft}^2)}{(0.8625\ \text{kip/ft}^2)}$$

$$Ratio = 0.99034$$

Status: **PASS**
Ratio: **0.990**






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

$H_o$ - Lateral force per length of pile,

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

$$H_o = \frac{(-3.311\text{ kip})}{1.57 \times (48\text{ in})}$$

$$H_o = -0.52723\text{ kip/ft}$$

$M_o$ - Moment per length of pile,

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

$$M_o = \frac{(34.779\text{ kipft}) + ((-3.311\text{ kip}) \times (0\text{ ft}))}{1.57 \times (48\text{ in})}$$

$$M_o = 5.5381\text{ kipft/ft}$$

$E$ - Distance from lateral load to resisting surface,

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

$$E = \frac{(5.5381\text{ kipft/ft})}{(-0.52723\text{ kip/ft})}$$

$$E = 10.504\text{ ft}$$

$a$ - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (5.5381\text{ kipft/ft}) \times (5.75\text{ ft})) + \left(3 \times (-0.52723\text{ kip/ft}) \times (5.75\text{ ft})^2\right)}{(6 \times (5.5381\text{ kipft/ft})) + (4 \times (-0.52723\text{ kip/ft}) \times (5.75\text{ ft}))}$$

$$a = 3.9614\text{ ft}$$

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

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

$$V_{max} = ((-0.52723\text{ kip/ft}) \times (48\text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (10.504\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(3.9614\text{ ft})}{(5.75\text{ ft})}\right)^2\right] + \left[4 \times \left(\frac{3 \times (10.504\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(3.9614\text{ ft})}{(5.75\text{ ft})}\right)^3\right]\right]$$

$$V_{max} = 8.2085\text{ kip}$$

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

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

$$M_{max} = ((-0.52723\text{ kip/ft}) \times (48\text{ in}) \times (5.75\text{ ft})) \times \left[\left(\frac{(10.504\text{ ft})}{(5.75\text{ ft})} + \frac{(3.9614\text{ ft})}{2 \times (5.75\text{ ft})}\right) - \left[\left(\frac{4 \times (10.504\text{ ft})}{(5.75\text{ ft})} + 3\right) \times \left(\frac{(3.9614\text{ ft})}{2 \times (5.75\text{ ft})}\right)^3\right] + \left[\left(\frac{3 \times (10.504\text{ ft})}{(5.75\text{ ft})} + 2\right) \times \left(\frac{(3.9614\text{ ft})}{2 \times (5.75\text{ ft})}\right)^4\right]\right]$$

$$M_{max} = 22.498 \text{ kipft}$$

**Minimum Reinforcement Check (LRFD)**

**Parameters:**

$f'_{ck} = 2.5 \text{ ksi}$ - Concrete strength,
$f_{yk} = 60 \text{ ksi}$ - Longitudinal reinforcement strength,
$\phi = 0.65$ - Reduction factor for axial strength,
*Table 22.4.2.1* $\alpha = 0.8$ - Alpha factor for axial strength,
$A_g = 2304 \text{ in}^2$ - Gross area of concrete,

**Longitudinal reinforcement:**

Required reinforcement due to axial load, $A_{st,required}$
*22.4.2.2, 10.6.1.1* $A_{st,required}$

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

$$A_{st,required} = Min\left[\frac{\frac{(7.392 \text{ kip})}{(0.65)\times(0.8)} - \left(0.85 \times (2.5 \text{ ksi}) \times \left(2304 \text{ in}^2\right)\right)}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, \left(0.08 \times \left(2304 \text{ in}^2\right)\right)\right]$$

$$A_{st,required} = -84.35 \text{ in}^2$$

$A_{min}$ - Governing minimum reinforcement area,

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

$$A_{min} = Max\left[\left(-84.35 \text{ in}^2\right), \left(0.0018 \times \left(2304 \text{ in}^2\right)\right)\right]$$

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

$n_{rebar}$ - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

$A_{st}$ - Actual total reinforcement area,

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

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

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

$Ratio$ - Capacity

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

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

$$Ratio = 0.96556$$

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

*25.2.3* $s_{rebar}$ - Minimum spacing of reinforcement,

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

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

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

**Ties:**

*25.7.2.2* Since longitudinal reinforcement is $\leq$ No. 10ø: Use #3(0.375 in)
*25.7.2.1* $s_{ties}$ - Maximum spacing of ties,

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

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

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

**Summary:**

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

Ties: **#3(0.375 in) - 10 in**

**Axial Compression Strength (ACI 318-19, LRFD)**

*22.4.2.2* $\phi P_N$ - Allowable axial compressive strength

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

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

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

$Ratio$ - Capacity

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

$$Ratio = \frac{(7.392\ \text{kip})}{(2675.2\ \text{kip})}$$

$$Ratio = 0.0027632$$

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

**Shear Strength (ACI 318-19, LRFD)**

**Parameters:**

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

$$d = 0.80\, D$$

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

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

*22.5.5.1.3* $\lambda_s$ - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

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

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

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

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

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$, $P = 7.392\ \text{kip} \rightarrow 7392\ \text{lbf}$,
*22.5.5.1.1(a)* $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500\ \text{psi})} + \frac{(7392\ \text{lbf})}{6 \times (2304\ \text{in}^2)}\right] \times (48\ \text{in}) \times (38.4\ \text{in})$$

$$V_{c,a} = 119.47\ \text{kip}$$

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

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

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

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

$V_c$ - Governing shear strength of concrete

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

$$V_c = Min\left[(296.21\ \text{kip}), (119.47\ \text{kip}), (348.89\ \text{kip})\right]$$

$$V_c = 119.47\ \text{kip}$$

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5\ \text{ksi} \rightarrow 2500\ \text{psi}$,

22.5.1.2 $V_{s,a}$ - Shear strength of steel (a)

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

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

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

$A_v$ - Ties rebar area,

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

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

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

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

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

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

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

$V_s$ - Governing shear strength of steel

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

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

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

22.5.1.1 $\phi V_n$ - Allowable shear strength

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

$$\phi V_n = (0.65) \times ((119.47\ \text{kip}) + (50.894\ \text{kip}))$$

$$\phi V_n = 110.74\ \text{kip}$$

**Considering x-direction:**

$V_{max} = 8.2085\ \text{kip}$ - Maximum shear force in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(8.2085\ \text{kip})}{(110.74\ \text{kip})}$$

$$Ratio = 0.074126$$

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

**Flexural Strength (ACI 318-19, LRFD)**

$S_m$ - Section modulus

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

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

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

$\lambda = 1$ - Concrete modification factor (Normal concrete),
Allowable flexural strength:
$M_n$ shall be the lesser of:
$\phi M_{n,1}$

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

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

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

14.5.2.1b $\phi M_{n,2}$

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

$$\phi M_{n,2} = \phi\, 0.85\, f_{ck}\, Z_m$$

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

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

Therefore,
$\phi M_n$ - Allowable flexural strength,

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

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

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

**Considering x-direction:**

$M_{max} = 22.498 \text{ kipft}$ - Maximum moment in the x-direction,
$Ratio$ - Capacity

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

$$Ratio = \frac{(22.498 \text{ kipft})}{(249.6 \text{ kipft})}$$

$$Ratio = 0.090135$$

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