

Your Project Calculations



Project Name: Walker Lake Small-CU2

S3D Model Link:
https://platform.skyciv.com/structural?preload_name=Walker%20Lake%20Small-CU2&preload_path=Shared%20Enterprise%20Folder/MT_Solar_Projects/3_2024

Public Model Link:
https://platform.skyciv.com/structural-viewer?project_id=GOOrAUznxystuSNvOKBySWw5Z55xDcoTpP7nFAFYAT8gc2nMyagVrVFTc5rd1QVYn

Array Specification

| | |
|-----------------------------|-------------------------------|
| Product: | Beam |
| Unique ID: | 2P-17-6TOP-SD-57-L-5Hx5W-IAJJ |
| Duty Classification: | SD |
| Module Width: | 41.50 in |
| Module Length: | 82.13in |
| Number of Rows: | 5 |
| Number of Columns: | 5 |
| Total Number of Modules: | 25 |
| Desired Tilt Angle: | 25 |
| Front Edge Clearance: | 10 |
| Total Array Height at Tilt: | 17.35 ft |
| Total Frame Length: | 34.00 ft |
| Frame Weight: | 1700 lbs |
| Array Dimensions N/S: | 17.50 ft |
| Array Dimensions E/W: | 34.64 ft |
| Rail Length: | 210.00 in |
| Rail Spacing: | 3.42 ft |
| Rail Check: | Not Checked |

Support Specifications

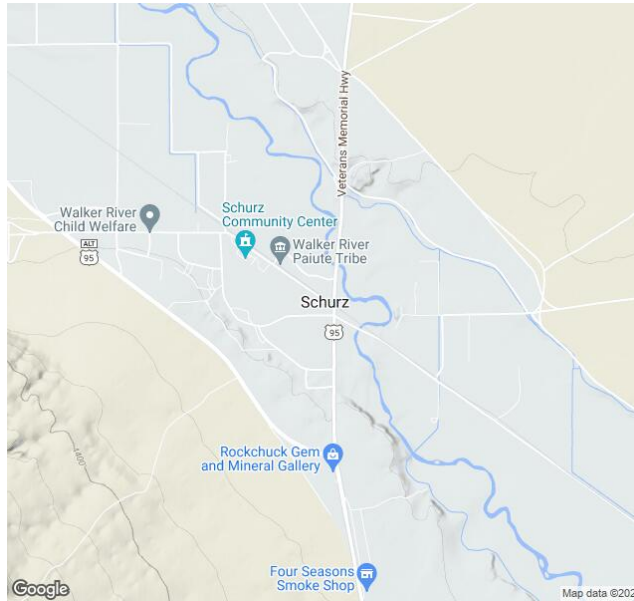
| | |
|--------------------------|-----------------|
| Pole Size: | 6in Pipe Sch 80 |
| Pole Length above Grade: | 13.70 ft |
| Number of Poles: | 2 |
| Pole Spacing: | 17 ft |

Foundation Specifications

| | |
|---------------------------------|------------------------------------|
| Foundation Type: | Square |
| Foundation Dimensions: | 48 x 48 in |
| Foundation Depth (below grade): | Pile 1: 6.25 ft Pile 2: 6.25 ft |
| Foundation Volume: | 7.407 y ³ |
| Foundation Result: | PASSED |
| Mount Twist: | 0.448788 kip |

Site Info

| | |
|----------------------------|--------------------------|
| Risk Category: | I |
| Exposure: | C |
| Soil Classification: | sand |
| Site Location: | M986+RQ, Schurz, NV, USA |
| Wind Speed: | 91 mph |
| Snow Load: | 10 psf |
| Design Uplift Pressure: | Multiple pressures |
| Design Downforce Pressure: | Multiple pressures |
| Design Snow Pressure: | 0.004948 ksf |



Design Disclaimer

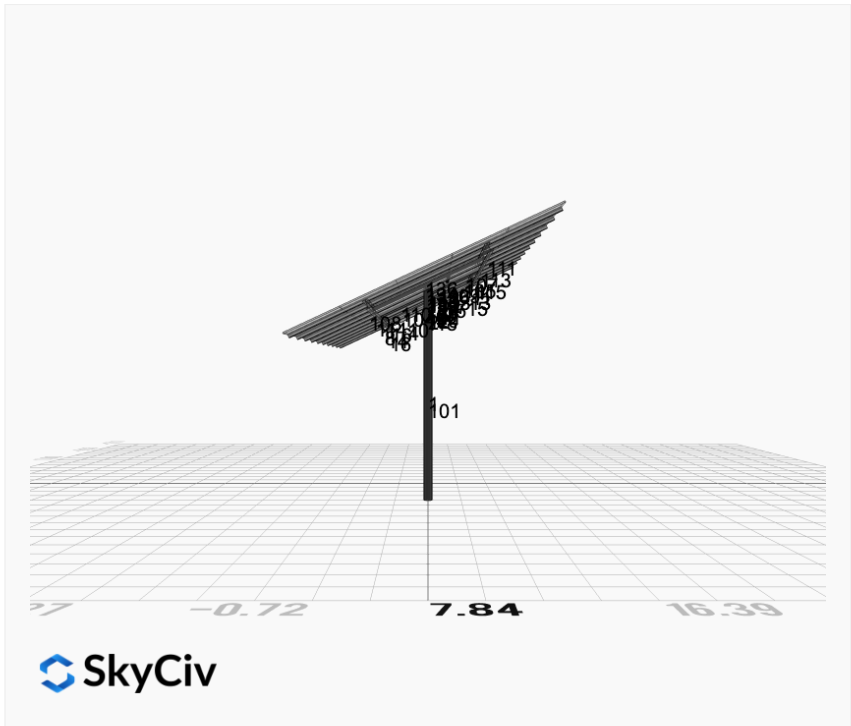
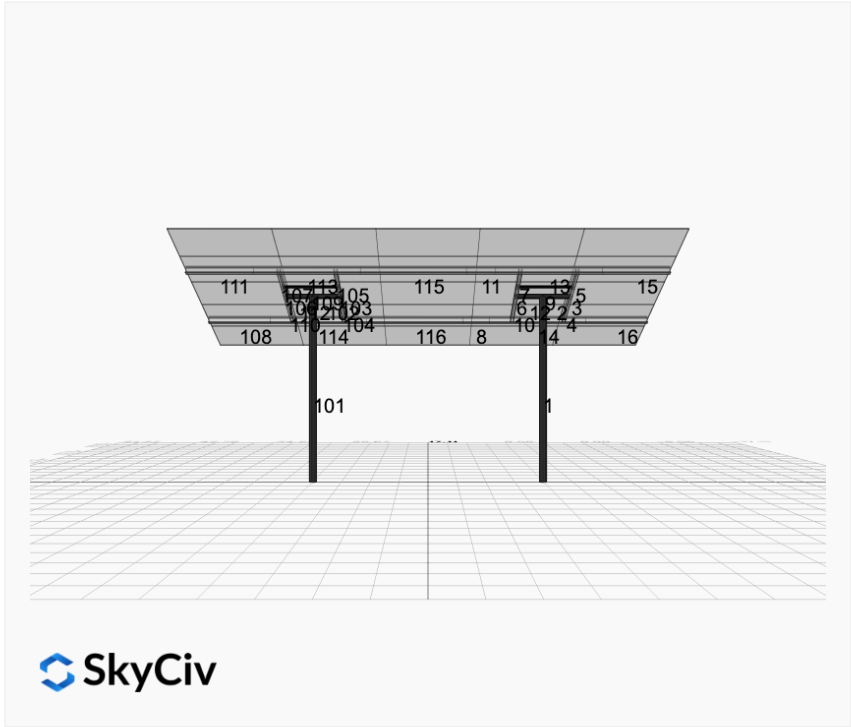
This software should be used for preliminary designs and should not be used as a final design unless reviewed, verified and designed by a qualified structural engineer.

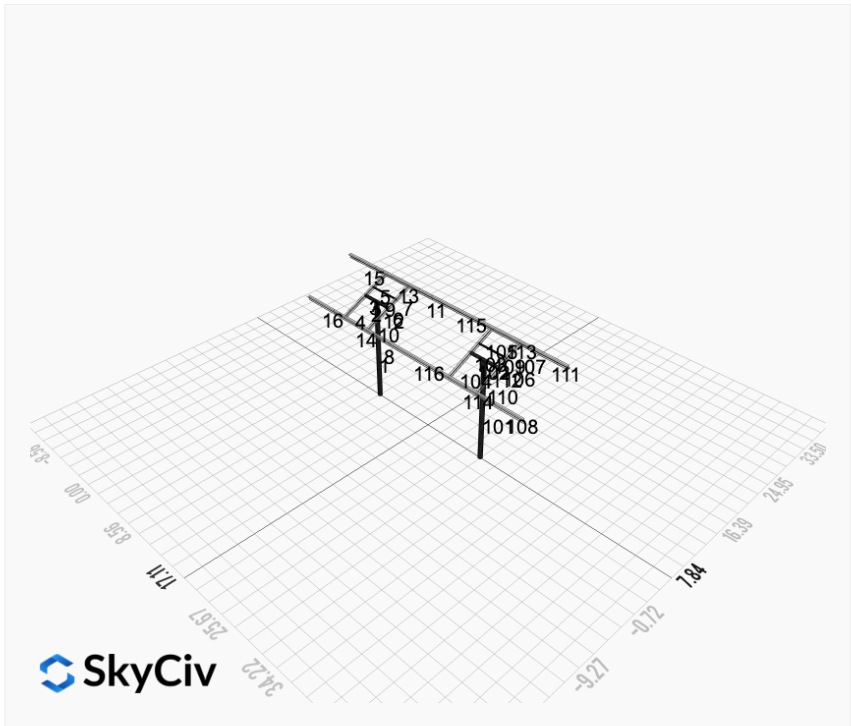
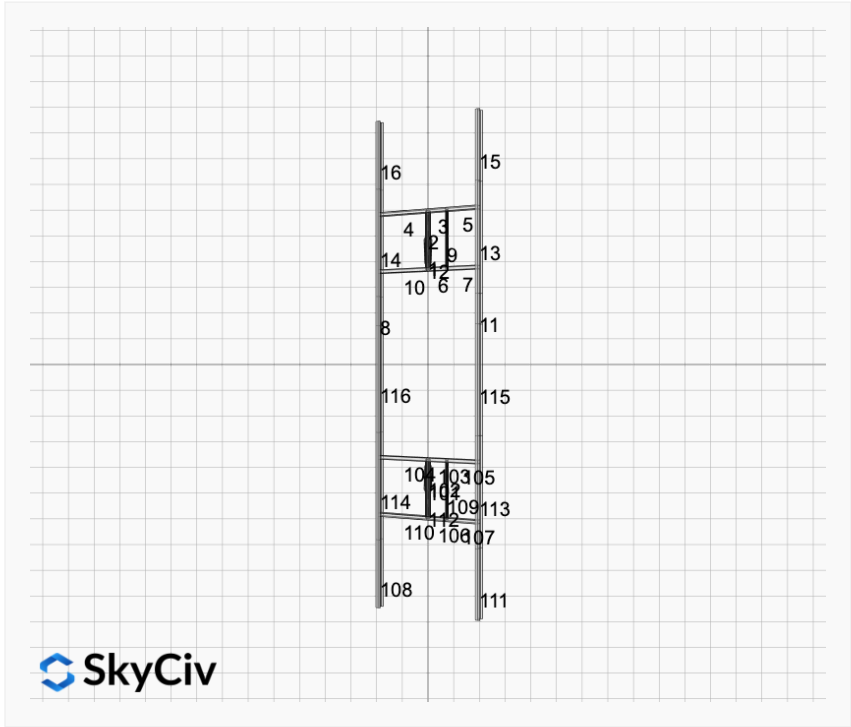
AutoDesigner Input

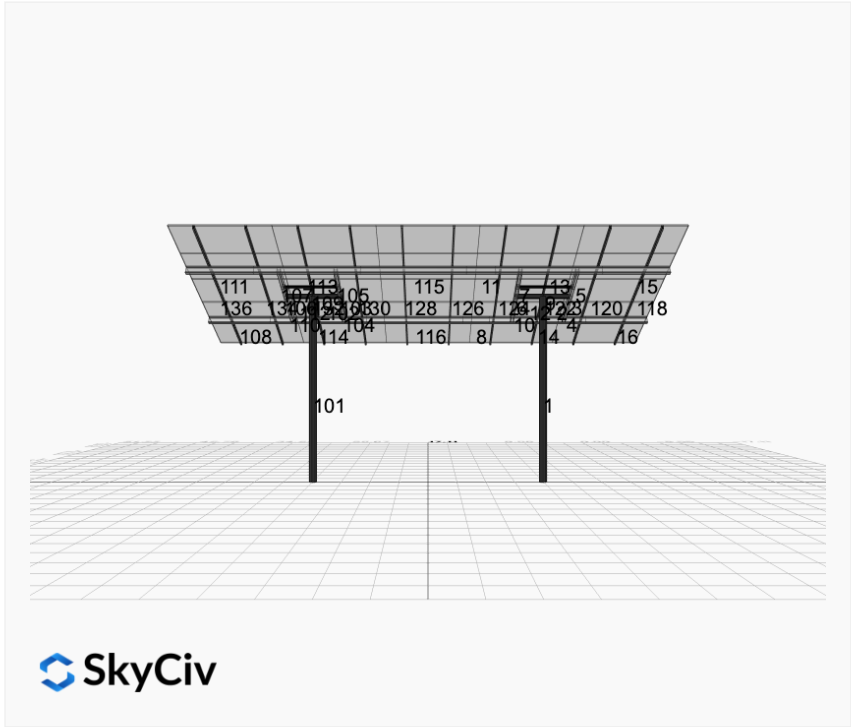
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Design Notes:

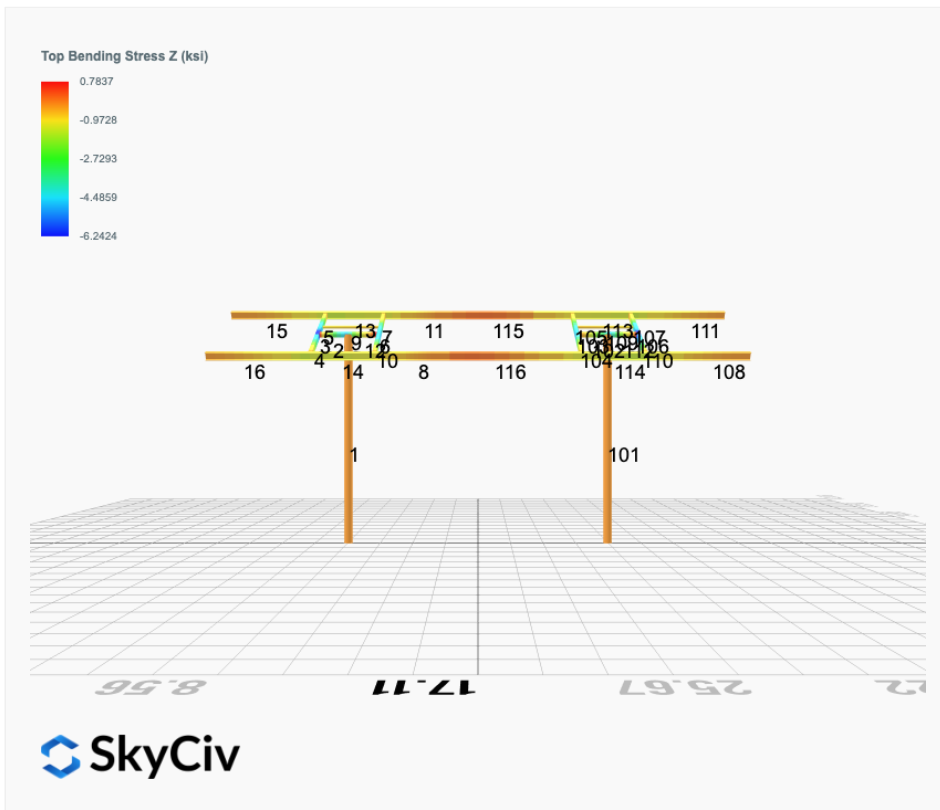
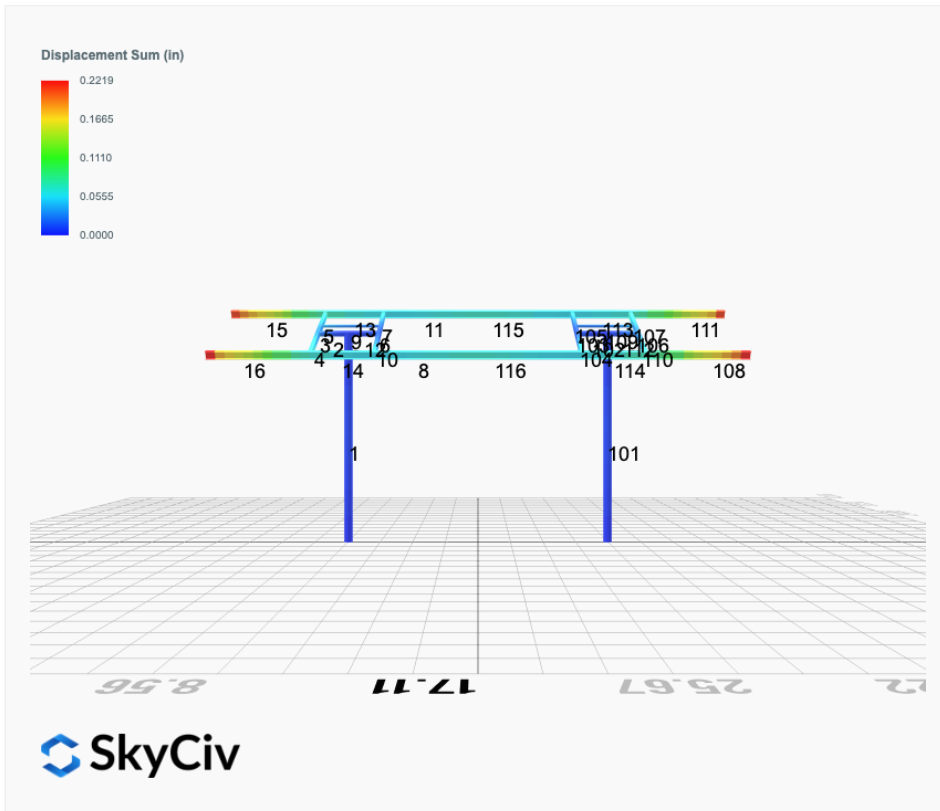
- AISC Deflection checks are set to L/1 due to structure design intent
- Foundation Soil Parameters used in this Autodesign are all estimates, proper geotechnical reports are required to confirm soil profiles
- Wind speeds, snow loads and other site specific results are based on ASCE 7 2016
- Steel frame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

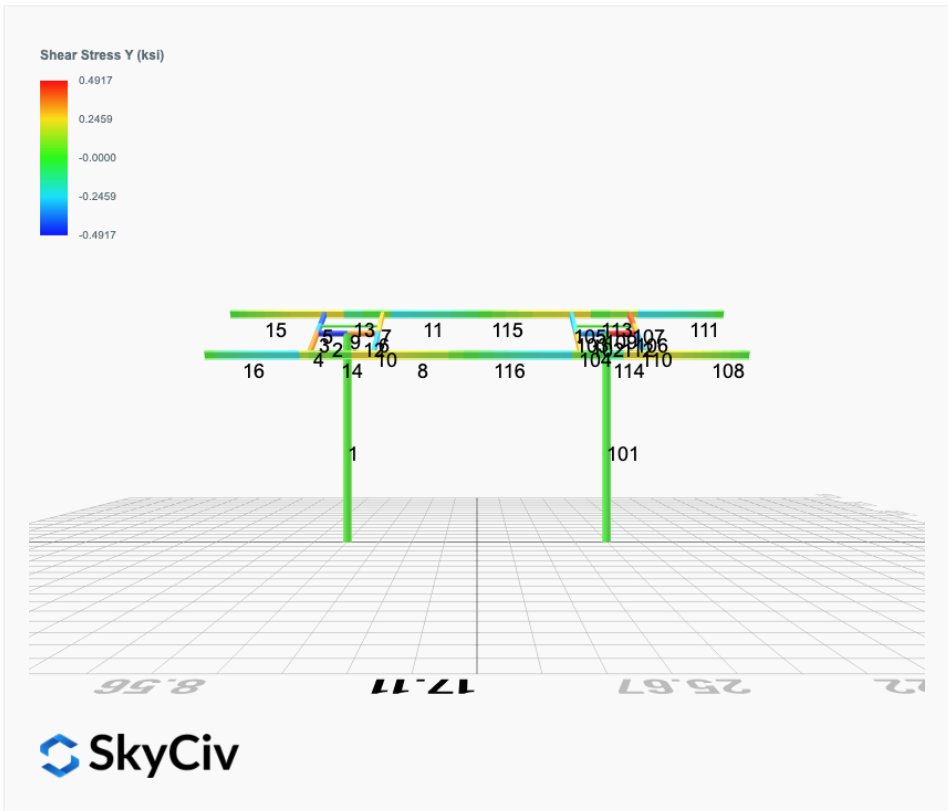
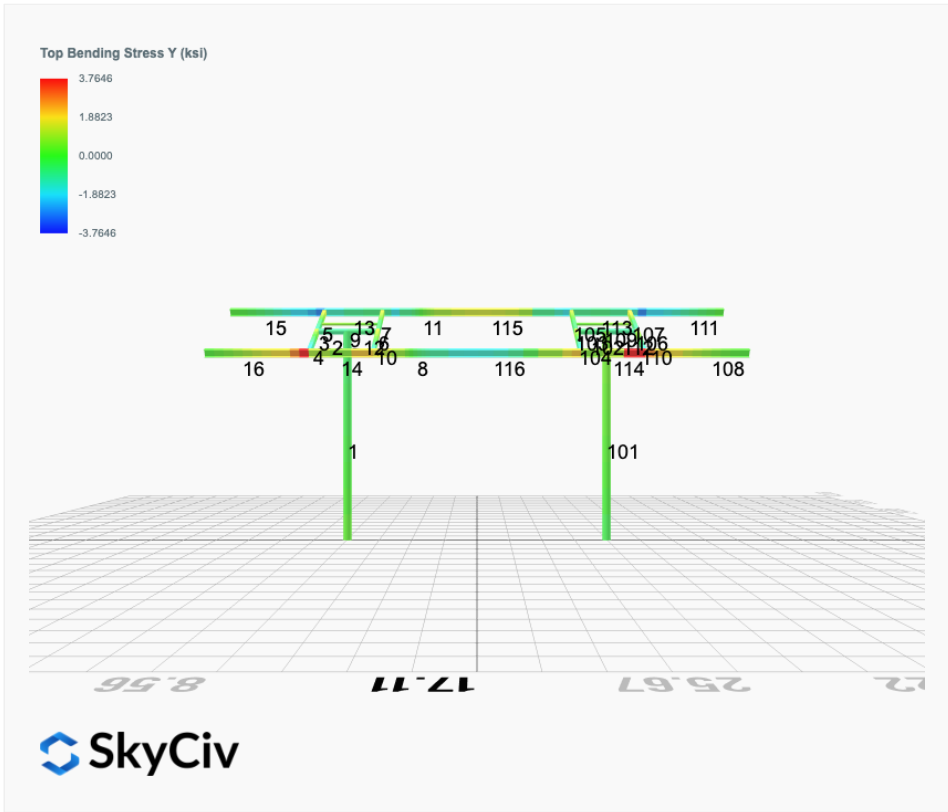


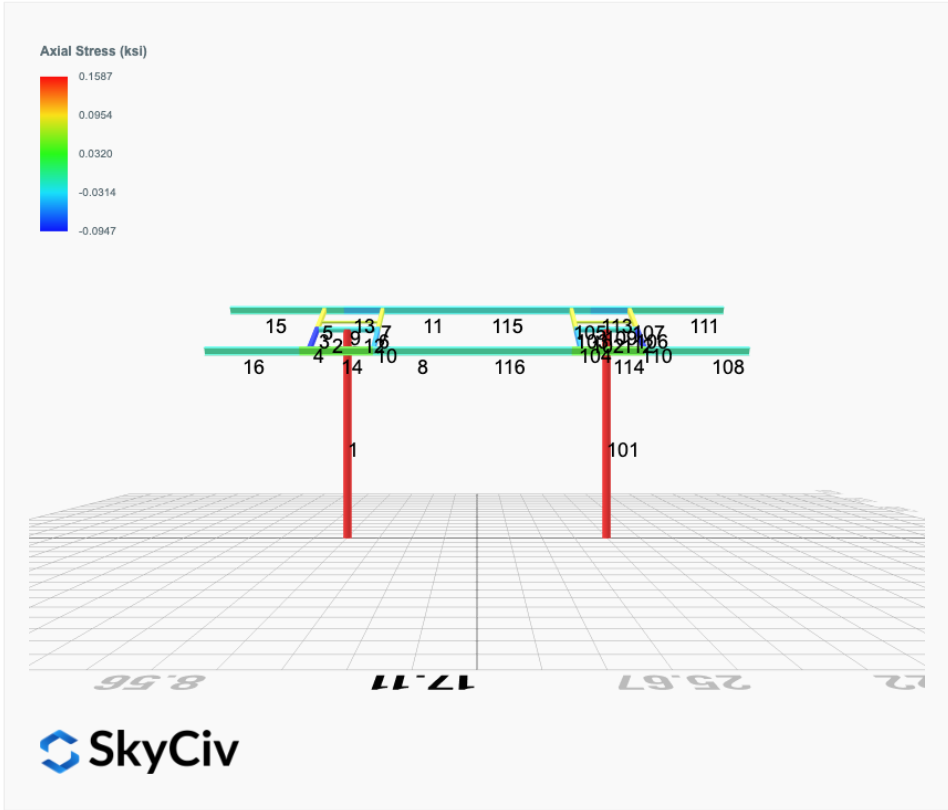




FEM Results (Envelope Worst Case for each member)







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 | 2.2123 | -0.0496 | -0.2146 | 0.0669 | 0.0218 |
| ULS: 2. D + L | 0.0000 | 2.2123 | -0.0496 | -0.2146 | 0.0669 | 0.0218 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 3.5465 | -0.0891 | -0.3854 | 0.1200 | 0.0243 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 2.2123 | -0.0496 | -0.2146 | 0.0669 | 0.0218 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 3.2130 | -0.0792 | -0.3427 | 0.1067 | 0.0237 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 2.2123 | -0.0496 | -0.2146 | 0.0669 | 0.0218 |
| ULS: 5b. D + 0.7E | 0.0000 | 2.2123 | -0.0496 | -0.2146 | 0.0669 | 0.0218 |
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | -0.0000 | 3.2130 | -0.0792 | -0.3427 | 0.1067 | 0.0237 |
| ULS: 8. 0.6D + 0.7E | 0.0000 | 1.3274 | -0.0298 | -0.1288 | 0.0401 | 0.0131 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -1.5869 | 5.6154 | -0.1686 | -0.7277 | 0.2720 | 23.0257 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -1.5869 | 5.6154 | -0.1686 | -0.7277 | 0.2720 | 23.0257 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 1.3885 | -0.7654 | 0.0540 | 0.2292 | -0.1125 | -18.0680 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 1.1902 | -0.3400 | 0.0400 | 0.1691 | -0.0881 | -23.1976 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.1902 | 5.7653 | -0.1684 | -0.7275 | 0.2606 | 17.2766 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.1902 | 5.7653 | -0.1684 | -0.7275 | 0.2606 | 17.2766 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0414 | 0.9797 | -0.0015 | -0.0098 | -0.0278 | -13.5436 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.8926 | 1.2987 | -0.0120 | -0.0549 | -0.0095 | -17.3908 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.1902 | 4.7646 | -0.1388 | -0.5994 | 0.2207 | 17.2747 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.1902 | 4.7646 | -0.1388 | -0.5994 | 0.2207 | 17.2747 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0414 | -0.0210 | 0.0281 | 0.1182 | -0.0676 | -13.5455 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.8926 | 0.2981 | 0.0176 | 0.0732 | -0.0494 | -17.3927 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -1.5869 | 4.7305 | -0.1487 | -0.6419 | 0.2453 | 23.0170 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -1.5869 | 4.7305 | -0.1487 | -0.6419 | 0.2453 | 23.0170 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 1.3885 | -1.6503 | 0.0739 | 0.3150 | -0.1392 | -18.0767 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 1.1902 | -1.2250 | 0.0598 | 0.2550 | -0.1148 | -23.2063 |

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 | 8.9937 |
| Shear X | -2.6448 |
| Shear Z | -0.2779 |
| Moment X | -1.2030 |
| Moment Y (Twist) | 0.4486 |
| Moment Z | 39.8198 |

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 | 5.7653 |
| Shear X | -1.5869 |
| Shear Z | -0.1686 |
| Moment X | -0.7277 |
| Moment Y (Twist) | 0.2720 |
| Moment Z | 23.2063 |

Reaction Forces for Foundation 2 (Node ID#101), (kip, kip-ft)

ASD Load Combination Results

| Name | Fx | Fy | Fz | Mx | My | Mz |
|--|---------|--------|--------|--------|---------|--------|
| ULS: 1. D | -0.0000 | 2.2123 | 0.0496 | 0.2146 | -0.0669 | 0.0218 |
| ULS: 2. D + L | -0.0000 | 2.2123 | 0.0496 | 0.2146 | -0.0669 | 0.0218 |
| ULS: 3. D + (S or Lr or R) | 0.0000 | 3.5465 | 0.0891 | 0.3854 | -0.1200 | 0.0243 |
| ULS: 3. D + (S or Lr or R) | -0.0000 | 2.2123 | 0.0496 | 0.2146 | -0.0669 | 0.0218 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | 0.0000 | 3.2130 | 0.0792 | 0.3427 | -0.1067 | 0.0237 |
| ULS: 4. D + 0.75L + 0.75(S or Lr or R) | -0.0000 | 2.2123 | 0.0496 | 0.2146 | -0.0669 | 0.0218 |
| ULS: 5b. D + 0.7E | -0.0000 | 2.2123 | 0.0496 | 0.2146 | -0.0669 | 0.0218 |

| Name | Fx | Fy | Fz | Mx | My | Mz |
|---|---------|---------|---------|---------|---------|----------|
| ULS: 6b. D + 0.75L + 0.75(0.7)E + 0.75S | 0.0000 | 3.2130 | 0.0792 | 0.3427 | -0.1067 | 0.0237 |
| ULS: 8. 0.6D + 0.7E | -0.0000 | 1.3274 | 0.0298 | 0.1288 | -0.0401 | 0.0131 |
| ULS: 5a. D + 0.6W_Wind downforce Case A only | -1.5869 | 5.6154 | 0.1686 | 0.7277 | -0.2720 | 23.0257 |
| ULS: 5a. D + 0.6W_Wind downforce Case B only | -1.5869 | 5.6154 | 0.1686 | 0.7277 | -0.2720 | 23.0257 |
| ULS: 5a. D + 0.6W_Wind uplift Case A only | 1.3885 | -0.7654 | -0.0540 | -0.2292 | 0.1125 | -18.0680 |
| ULS: 5a. D + 0.6W_Wind uplift Case B only | 1.1902 | -0.3400 | -0.0400 | -0.1691 | 0.0881 | -23.1976 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.1902 | 5.7653 | 0.1684 | 0.7275 | -0.2606 | 17.2766 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.1902 | 5.7653 | 0.1684 | 0.7275 | -0.2606 | 17.2766 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0414 | 0.9797 | 0.0015 | 0.0099 | 0.0278 | -13.5436 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.8926 | 1.2987 | 0.0120 | 0.0549 | 0.0095 | -17.3908 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case A only | -1.1902 | 4.7646 | 0.1388 | 0.5995 | -0.2207 | 17.2747 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind downforce Case B only | -1.1902 | 4.7646 | 0.1388 | 0.5995 | -0.2207 | 17.2747 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case A only | 1.0414 | -0.0210 | -0.0281 | -0.1182 | 0.0676 | -13.5455 |
| ULS: 6a. D + 0.75L + 0.75(0.6)W + 0.75(S or Lr or R)_Wind uplift Case B only | 0.8926 | 0.2981 | -0.0176 | -0.0732 | 0.0494 | -17.3927 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case A only | -1.5869 | 4.7305 | 0.1487 | 0.6419 | -0.2453 | 23.0170 |
| ULS: 7. 0.6D + 0.6W_Wind downforce Case B only | -1.5869 | 4.7305 | 0.1487 | 0.6419 | -0.2453 | 23.0170 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case A only | 1.3885 | -1.6503 | -0.0739 | -0.3150 | 0.1392 | -18.0767 |
| ULS: 7. 0.6D + 0.6W_Wind uplift Case B only | 1.1902 | -1.2250 | -0.0598 | -0.2550 | 0.1148 | -23.2063 |

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 | 8.9937 |
| Shear X | -2.6448 |
| Shear Z | 0.2779 |
| Moment X | 1.2030 |
| Moment Y (Twist) | 0.4488 |
| Moment Z | 39.8204 |

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 | 5.7653 |
| Shear X | -1.5869 |
| Shear Z | 0.1686 |
| Moment X | 0.7277 |
| Moment Y (Twist) | 0.2720 |
| Moment Z | 23.2063 |

Project Details

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

User Name: sales@mtsolar.us
 Project Name: Walker Lake Small-CU2
 Unit System: imperial



Design Input Information

| Design Factors | | | |
|----------------|----------|----------|----------|
| Φ_t | Φ_c | Φ_b | Φ_v |
| 0.9 | 0.9 | 0.9 | 0.9 |

| Design Materials | | | |
|------------------|---------|----------------------|----------------------|
| ID | E (ksi) | F _y (ksi) | F _u (ksi) |
| 1 | 29000 | 50 | 65 |

Section Dimensions

| ID | Name | d (in) | t _w (in) | | | | |
|----|-----------------|--------|---------------------|--|--|--|--|
| 1 | 2in Pipe Sch 40 | 2.38 | 0.15 | | | | |
| 4 | 4in Pipe Sch 40 | 4.50 | 0.24 | | | | |
| 8 | 6in Pipe Sch 80 | 6.63 | 0.43 | | | | |

| ID | Name | d (in) | b (in) | t _w (in) | t _b (in) | r (in) | |
|----|------------|--------|--------|---------------------|---------------------|--------|--|
| 15 | HSS5x3x1/8 | 5.00 | 3.00 | 0.12 | 0.12 | 0.12 | |

| ID | Name | d (in) | t _w (in) | b _t (in) | b _b (in) | t _t (in) | t _b (in) | r (in) |
|----|------|--------|---------------------|---------------------|---------------------|---------------------|---------------------|--------|
| 18 | W6x9 | 5.90 | 0.17 | 3.94 | 3.94 | 0.21 | 0.21 | 0.25 |

| Section Properties | | | | | | | | |
|--------------------|-----------------|----------------------|----------------------|------------------------------------|------------------------------------|-----------------------------------|------------------------------------|------------------------------------|
| ID | Name | A (in ²) | J (in ⁴) | I _{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 |

| | | | | | | |
|-----|--------|-------|-------|------|-------|-------|
| 115 | 120.60 | 89.27 | 23.18 | 6.45 | 30.09 | 45.74 |
| 116 | 120.60 | 89.27 | 19.97 | 6.45 | 30.09 | 45.74 |

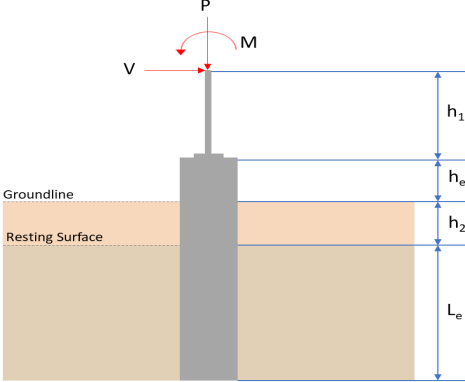
Design Ratio

| Member ID | P | M _z | M _y | V _y | V _z | (P,M _z ,M _y) | Worst LC | KL/r | δ | Status |
|-----------|-------|----------------|----------------|----------------|----------------|-------------------------------------|----------|--------------|--------------|--------|
| 1 | 0.117 | 0.640 | 0.042 | 0.023 | 0.002 | 0.712 | #13 | 0.786 | Not Required | Pass |
| 2 | 0.001 | 0.553 | 0.184 | 0.111 | 0.033 | 0.737 | #13 | 0.052 | Not Required | Pass |
| 3 | 0.007 | 0.838 | 0.085 | 0.086 | 0.011 | 0.905 | #13 | 0.044 | Not Required | Pass |
| 4 | 0.007 | 0.816 | 0.155 | 0.082 | 0.019 | 0.839 | #13 | 0.078 | Not Required | Pass |
| 5 | 0.007 | 0.519 | 0.167 | 0.084 | 0.024 | 0.549 | #13 | 0.073 | Not Required | Pass |
| 6 | 0.006 | 0.681 | 0.046 | 0.067 | 0.003 | 0.699 | #13 | 0.044 | Not Required | Pass |
| 7 | 0.006 | 0.423 | 0.112 | 0.068 | 0.017 | 0.436 | #13 | 0.073 | Not Required | Pass |
| 8 | 0.001 | 0.069 | 0.035 | 0.040 | 0.005 | 0.087 | #13 | 0.088 | Not Required | Pass |
| 9 | 0.005 | 0.106 | 0.057 | 0.003 | 0.002 | 0.165 | #13 | 0.198 | Not Required | Pass |
| 10 | 0.005 | 0.660 | 0.140 | 0.067 | 0.019 | 0.762 | #13 | 0.078 | Not Required | Pass |
| 11 | 0.001 | 0.069 | 0.036 | 0.041 | 0.005 | 0.085 | #13 | 0.059 | Not Required | Pass |
| 12 | 0.002 | 0.392 | 0.157 | 0.088 | 0.029 | 0.550 | #13 | 0.052 | Not Required | Pass |
| 13 | 0.003 | 0.280 | 0.169 | 0.056 | 0.007 | 0.390 | #21 | 0.177 | Not Required | Pass |
| 14 | 0.003 | 0.279 | 0.169 | 0.054 | 0.007 | 0.386 | #21 | 0.177 | Not Required | Pass |
| 15 | 0.000 | 0.125 | 0.090 | 0.041 | 0.005 | 0.189 | #21 | Not Required | Not Required | Pass |
| 16 | 0.000 | 0.122 | 0.090 | 0.040 | 0.005 | 0.187 | #21 | Not Required | Not Required | Pass |
| 101 | 0.117 | 0.640 | 0.042 | 0.023 | 0.002 | 0.712 | #13 | 0.786 | Not Required | Pass |
| 102 | 0.002 | 0.392 | 0.157 | 0.088 | 0.029 | 0.550 | #13 | 0.052 | Not Required | Pass |
| 103 | 0.006 | 0.681 | 0.046 | 0.067 | 0.003 | 0.699 | #13 | 0.044 | Not Required | Pass |
| 104 | 0.005 | 0.660 | 0.140 | 0.067 | 0.019 | 0.762 | #13 | 0.078 | Not Required | Pass |
| 105 | 0.006 | 0.423 | 0.112 | 0.068 | 0.017 | 0.436 | #13 | 0.073 | Not Required | Pass |
| 106 | 0.007 | 0.839 | 0.085 | 0.086 | 0.011 | 0.905 | #13 | 0.044 | Not Required | Pass |
| 107 | 0.007 | 0.519 | 0.167 | 0.084 | 0.024 | 0.549 | #13 | 0.073 | Not Required | Pass |
| 108 | 0.000 | 0.122 | 0.090 | 0.040 | 0.005 | 0.187 | #21 | Not Required | Not Required | Pass |
| 109 | 0.005 | 0.106 | 0.057 | 0.003 | 0.002 | 0.165 | #13 | 0.198 | Not Required | Pass |
| 110 | 0.007 | 0.816 | 0.155 | 0.082 | 0.019 | 0.839 | #13 | 0.078 | Not Required | Pass |
| 111 | 0.000 | 0.125 | 0.090 | 0.041 | 0.005 | 0.189 | #21 | Not Required | Not Required | Pass |
| 112 | 0.001 | 0.553 | 0.184 | 0.111 | 0.033 | 0.737 | #13 | 0.052 | Not Required | Pass |
| 113 | 0.003 | 0.280 | 0.169 | 0.056 | 0.007 | 0.390 | #21 | 0.177 | Not Required | Pass |
| 114 | 0.003 | 0.279 | 0.169 | 0.054 | 0.007 | 0.386 | #21 | 0.265 | Not Required | Pass |
| 115 | 0.001 | 0.069 | 0.065 | 0.041 | 0.005 | 0.109 | #21 | 0.214 | Not Required | Pass |
| 116 | 0.002 | 0.069 | 0.064 | 0.040 | 0.005 | 0.106 | #21 | 0.321 | 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 _n | 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: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_n) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.765</td> <td>8.994</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.587</td> <td>-2.645</td> </tr> <tr> <td>V_z (kip)</td> <td>-0.169</td> <td>-0.278</td> </tr> <tr> <td>M_x (kipft)</td> <td>-0.728</td> <td>-1.203</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.206</td> <td>39.820</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_n) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 5.765 | 8.994 | V_x (kip) | -1.587 | -2.645 | V_z (kip) | -0.169 | -0.278 | M_x (kipft) | -0.728 | -1.203 | M_z (kipft) | 23.206 | 39.820 | |
| Layer | Label | Allowable Bearing Pressure (q_n) (psf) | Allowable Lateral Pressure (R) (psf/ft) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | | | | | | | | | | | | | | | | | | | | | | | | | |
| Load Component | ASD | LRFD | | | | | | | | | | | | | | | | | | | | | | | | | | |
| P (kip) | 5.765 | 8.994 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -1.587 | -2.645 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | -0.169 | -0.278 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | -0.728 | -1.203 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 23.206 | 39.820 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.587 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25271 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(23.206 \text{ kipft}) + ((-1.587 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.6952 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9065 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.026911 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.728 \text{ kipft}) + ((-0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11592 \text{ kipft/ft}$$

Required depth of embedment in earth:

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

Solving the cubic equation:

$L_{e,z} = 1.8462 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9065 \text{ ft}), (1.8462 \text{ ft})]$$

$$L_{e,req} = 5.907 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.907 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94512$$

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

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

$$q = \frac{(5.765 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36031 \text{ kip/ft}^2$$

$$q = 0.00051 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36031 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18016$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25271 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.6952 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.6952 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.6952 \text{ kipft/ft})) + (4 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2822 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6952 \text{ kipft/ft})) + (3 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.6952 \text{ kipft/ft})) + (2 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.24429 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.6952 \text{ kipft/ft})) + ((-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89257 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2822 \text{ ft})}{2}$$

$$p_a = 0.32116 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24429 \text{ kip/ft}^2)}{(0.32116 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76063$$

p_a - Allowable lateral soil pressure at depth L_e ,

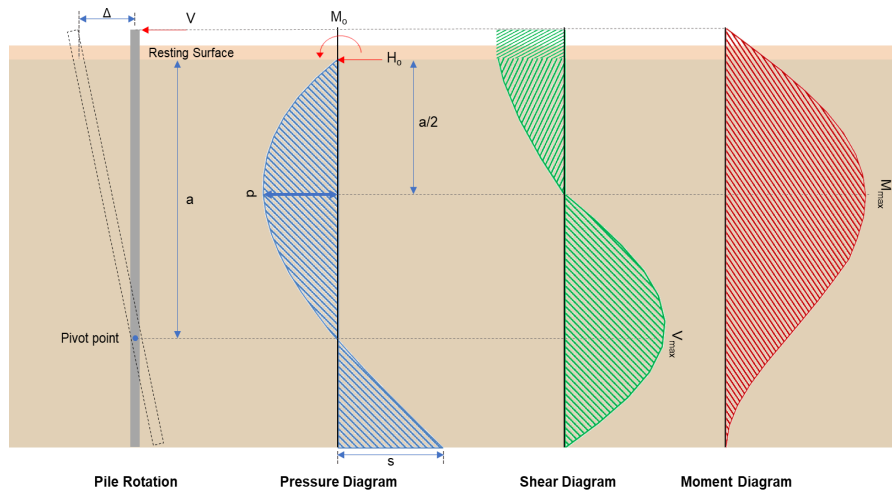
Status: **PASS**
Ratio: **0.760**

| | | |
|--|--|--|
| | $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89257 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95208$ | <p>Status: PASS Ratio: 0.950</p> |
| | <p>Considering z-direction:</p> <p>$H_o = -0.026911 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11592 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11592 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.026911 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.11592 \text{ kipft/ft})) + (4 \times (-0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4227 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.11592 \text{ kipft/ft})) + (3 \times (-0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.11592 \text{ kipft/ft})) + (2 \times (-0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.0028187 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.11592 \text{ kipft/ft})) + ((-0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.0097773 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4227 \text{ ft})}{2}$ $p_a = 0.33171 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.0028187 \text{ kip/ft}^2)}{(0.33171 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.0084977$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ | <p>Status: PASS Ratio: 0.010</p> |

$$Ratio = \frac{(0.0097773 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.010429$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.645 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42118 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.82 \text{ kipft}) + ((-2.645 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3408 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3408 \text{ kipft/ft})}{(-0.42118 \text{ kip/ft})}$$

$$E = 15.055 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3408 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.42118 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.3408 \text{ kipft/ft})) + (4 \times (-0.42118 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2796 \text{ ft}$$

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

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

$$V_{max} = ((-0.42118 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2796 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2796 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2956 \text{ kip}$$

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

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

$$M_{max} = ((-0.42118 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(15.055 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 24.964 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-0.278 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.044268 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.203 \text{ kipft}) + ((-0.278 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19156 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19156 \text{ kipft/ft})}{(-0.044268 \text{ kip/ft})}$$

$$E = 4.3273 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19156 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.044268 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19156 \text{ kipft/ft})) + (4 \times (-0.044268 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4222 \text{ ft}$$

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

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

$$V_{max} = ((-0.044268 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4222 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4222 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33437 \text{ kip}$$

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

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

$$M_{max} = ((-0.044268 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.3273 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.94573 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.994 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.297 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.297 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

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

25.7.2.1 s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.994 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.003362$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.994 \text{ kip} \rightarrow 8994 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8994 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.68 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

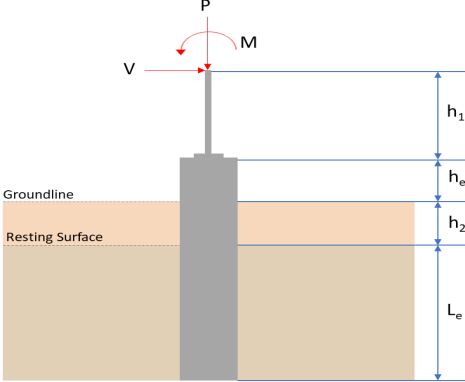
$$V_c = \text{Min}[V_{c,max}, V_{c,a}, V_{c,b}]$$

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.68 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.68 \text{ kip}$$

| | | |
|-----------------|---|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.68 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.88 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.2956 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2956 \text{ kip})}{(110.88 \text{ kip})}$ $\text{Ratio} = 0.074819$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33437 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33437 \text{ kip})}{(110.88 \text{ kip})}$ $\text{Ratio} = 0.0030157$ | <p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
|------------------|--|---|
| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 24.964 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.964 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.10002$ | <p>Status: PASS Ratio: 0.100</p> |
| | <p>Considering z-direction: $M_{max} = 0.94573 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.94573 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.003789$ | <p>Status: PASS Ratio: 0.000</p> |

| REFERENCES | CALCULATIONS | RESULTS | | | | | | | | | | | | | | | | | | | | | | | | | | |
|----------------|--|--|---|--|---|---|---|----------|---------|----------------|-----|------|-----------|-------|-------|-------------|--------|--------|-------------|-------|-------|---------------|-------|-------|---------------|--------|--------|--|
| | <p>SkyCiv Foundation Design Pile Foundation</p> <p>Design Information : Design code : IBC 2021 (International Building Code) Unit System : Imperial</p> | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <p>Pile Input</p>  <p>Geometry Pile shape: rectangular $b = 48$ in - Pile width $D = 48$ in - Pile depth $L = 6.25$ ft - Total pile length $h_1 = 0$ ft - Lateral load height from the top of the pile, $h_2 = 0$ ft - Depth to resting surface $h_e = 0$ ft - Length of pile above the ground</p> <p>Tabulation of Soil Parameters</p> <table border="1" data-bbox="416 1102 1193 1191"> <thead> <tr> <th>Layer</th> <th>Label</th> <th>Allowable Bearing Pressure (q_a) (psf)</th> <th>Allowable Lateral Pressure (R) (psf/ft)</th> </tr> </thead> <tbody> <tr> <td>1</td> <td>Sand, silty sand, clayey sand, silty gravel & clayey gravel</td> <td>2000.000</td> <td>150.000</td> </tr> </tbody> </table> <p>Tabulation of Loads</p> <table border="1" data-bbox="676 1285 935 1458"> <thead> <tr> <th>Load Component</th> <th>ASD</th> <th>LRFD</th> </tr> </thead> <tbody> <tr> <td>P (kip)</td> <td>5.765</td> <td>8.994</td> </tr> <tr> <td>V_x (kip)</td> <td>-1.587</td> <td>-2.645</td> </tr> <tr> <td>V_z (kip)</td> <td>0.169</td> <td>0.278</td> </tr> <tr> <td>M_x (kipft)</td> <td>0.728</td> <td>1.203</td> </tr> <tr> <td>M_z (kipft)</td> <td>23.206</td> <td>39.820</td> </tr> </tbody> </table> <p>Material Properties $f'_{ck} = 2.5$ ksi - Concrete strength,</p> | Layer | Label | Allowable Bearing Pressure (q_a) (psf) | Allowable Lateral Pressure (R) (psf/ft) | 1 | Sand, silty sand, clayey sand, silty gravel & clayey gravel | 2000.000 | 150.000 | Load Component | ASD | LRFD | P (kip) | 5.765 | 8.994 | V_x (kip) | -1.587 | -2.645 | V_z (kip) | 0.169 | 0.278 | M_x (kipft) | 0.728 | 1.203 | M_z (kipft) | 23.206 | 39.820 | |
| 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) | 5.765 | 8.994 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_x (kip) | -1.587 | -2.645 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| V_z (kip) | 0.169 | 0.278 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_x (kipft) | 0.728 | 1.203 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| M_z (kipft) | 23.206 | 39.820 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | <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> $H_o = \frac{V_x}{1.57 D}$ $H_o = \frac{(-1.587 \text{ kip})}{1.57 \times (48 \text{ in})}$ $H_o = -0.25271 \text{ kip/ft}$ <p>M_o - Moment per length of pile,</p> $M_o = \frac{M_z + (V_x H)}{1.57 D}$ | | | | | | | | | | | | | | | | | | | | | | | | | | | |

$$M_o = \frac{(23.206 \text{ kipft}) + ((-1.587 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 3.6952 \text{ kipft/ft}$$

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:

$L_{e,x} = 5.9065 \text{ ft}$ - Required depth in x-direction,

Considering z-direction:

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.169 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.026911 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(0.728 \text{ kipft}) + ((0.169 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.11592 \text{ kipft/ft}$$

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:

$L_{e,z} = 2.356 \text{ ft}$ - Required depth in z-direction,

Minimum embedded depth required:

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

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

$$L_{e,req} = \text{MAX}[(5.9065 \text{ ft}), (2.356 \text{ ft})]$$

$$L_{e,req} = 5.907 \text{ ft}$$

L_e - Actual embedded length of pile,

$$L_e = L - h_e - h_2$$

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

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

Ratio - Embedded depth

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

$$\text{Ratio} = \frac{(5.907 \text{ ft})}{(6.25 \text{ ft})}$$

$$\text{Ratio} = 0.94512$$

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

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

$$q = \frac{(5.765 \text{ kip})}{(16 \text{ ft}^2)}$$

$$q = 0.36031 \text{ kip/ft}^2$$

$$q = 0.00051 \text{ kip/ft}$$

Check bearing capacity ratio:

Ratio - Capacity

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

$$\text{Ratio} = \frac{(0.36031 \text{ kip/ft}^2)}{(2000 \text{ psf})}$$

$$\text{Ratio} = 0.18016$$

Status: **PASS**
Ratio: **0.180**

Czerniak

Lateral Soil Pressure (ASD):

L/D - Length to least lateral dimension ratio,

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

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

$$L/D = 1.5625$$

Since $L/D \leq 10$,

Pile is short.

Considering x-direction:

$H_o = -0.25271 \text{ kip/ft}$ - Lateral force per length of pile,

$M_o = 3.6952 \text{ kipft/ft}$ - Overturning moment per length of pile,

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (3.6952 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (3.6952 \text{ kipft/ft})) + (4 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2822 \text{ ft}$$

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

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

$$p = \frac{0.75 \times [(4 \times (3.6952 \text{ kipft/ft})) + (3 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 \times [(3 \times (3.6952 \text{ kipft/ft})) + (2 \times (-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$$

$$p = 0.24429 \text{ kip/ft}^2$$

s - Earth pressure against the pile at distance L_e ,

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

$$s = \frac{6 \times [(2 \times (3.6952 \text{ kipft/ft})) + ((-0.25271 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$$

$$s = 0.89257 \text{ kip/ft}^2$$

Check lateral soil pressure capacity:

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

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

$$p_a = (150 \text{ psf/ft}) \times \frac{(4.2822 \text{ ft})}{2}$$

$$p_a = 0.32116 \text{ kip/ft}^2$$

Ratio - Lateral soil capacity

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

$$\text{Ratio} = \frac{(0.24429 \text{ kip/ft}^2)}{(0.32116 \text{ kip/ft}^2)}$$

$$\text{Ratio} = 0.76063$$

p_a - Allowable lateral soil pressure at depth L_e ,

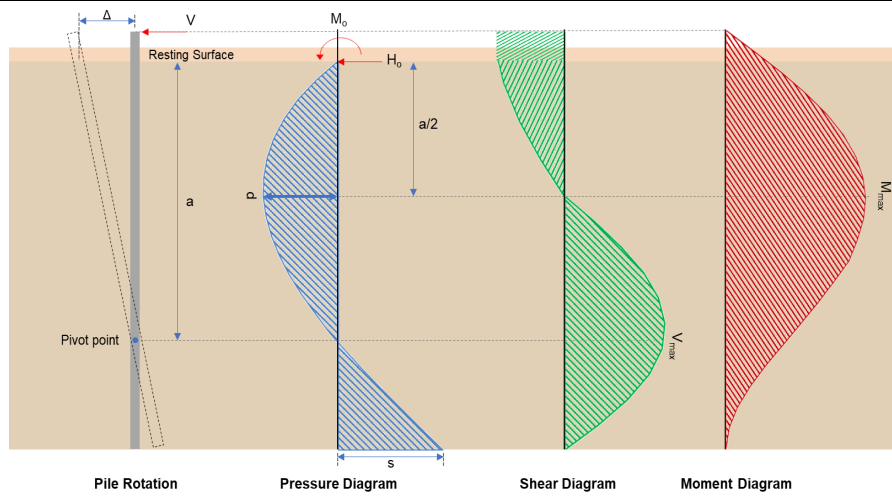
Status: **PASS**
Ratio: **0.760**

| | | |
|--|--|--|
| | $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ $\text{Ratio} = \frac{(0.89257 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.95208$ | Status: PASS Ratio: 0.950 |
| | <p>Considering z-direction:</p> <p>$H_o = 0.026911 \text{ kip/ft}$ - Lateral force per length of pile, $M_o = 0.11592 \text{ kipft/ft}$ - Overturning moment per length of pile, a - Distance from resting surface to pivot point,</p> $a = \frac{(4 M_o L_e) + (3 H_o L_e^2)}{(6 M_o) + (4 H_o L_e)}$ $a = \frac{(4 \times (0.11592 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.026911 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.11592 \text{ kipft/ft})) + (4 \times (0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))}$ $a = 4.4227 \text{ ft}$ <p>p - Earth pressure against the pile at distance $a/2$ from resting surface,</p> $p = \frac{0.75 [(4 M_o) + (3 H_o L_e)]^2}{L_e^2 [(3 M_o) + (2 H_o L_e)]}$ $p = \frac{0.75 [(4 \times (0.11592 \text{ kipft/ft})) + (3 \times (0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]^2}{(6.25 \text{ ft})^2 [(3 \times (0.11592 \text{ kipft/ft})) + (2 \times (0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]}$ $p = 0.026311 \text{ kip/ft}^2$ <p>s - Earth pressure against the pile at distance L_e,</p> $s = \frac{6 [(2 M_o) + (H_o L_e)]}{L_e^2}$ $s = \frac{6 [(2 \times (0.11592 \text{ kipft/ft})) + ((0.026911 \text{ kip/ft}) \times (6.25 \text{ ft}))]}{(6.25 \text{ ft})^2}$ $s = 0.061446 \text{ kip/ft}^2$ <p>Check lateral soil pressure capacity:</p> <p>p_a - Allowable lateral soil pressure at depth $a/2$,</p> $p_a = R \frac{a}{2}$ $p_a = (150 \text{ psf/ft}) \times \frac{(4.4227 \text{ ft})}{2}$ $p_a = 0.33171 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{p}{p_a}$ $\text{Ratio} = \frac{(0.026311 \text{ kip/ft}^2)}{(0.33171 \text{ kip/ft}^2)}$ $\text{Ratio} = 0.079321$ <p>p_s - Allowable lateral soil pressure at depth L_e,</p> $p_s = R L_e$ $p_s = (150 \text{ psf/ft}) \times (6.25 \text{ ft})$ $p_s = 0.9375 \text{ kip/ft}^2$ <p>Ratio - Lateral soil capacity</p> $\text{Ratio} = \frac{s}{p_s}$ | Status: PASS Ratio: 0.080 |

$$Ratio = \frac{(0.061446 \text{ kip/ft}^2)}{(0.9375 \text{ kip/ft}^2)}$$

$$Ratio = 0.065543$$

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



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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(-2.645 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = -0.42118 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(39.82 \text{ kipft}) + ((-2.645 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 6.3408 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(6.3408 \text{ kipft/ft})}{(-0.42118 \text{ kip/ft})}$$

$$E = 15.055 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (6.3408 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (-0.42118 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (6.3408 \text{ kipft/ft})) + (4 \times (-0.42118 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.2796 \text{ ft}$$

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

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

$$V_{max} = ((-0.42118 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2796 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2796 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 8.2956 \text{ kip}$$

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

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

$$M_{max} = ((-0.42118 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(15.055 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (15.055 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.2796 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 24.964 \text{ kipft}$$

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

H_o - Lateral force per length of pile,

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

$$H_o = \frac{(0.278 \text{ kip})}{1.57 \times (48 \text{ in})}$$

$$H_o = 0.044268 \text{ kip/ft}$$

M_o - Moment per length of pile,

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

$$M_o = \frac{(1.203 \text{ kipft}) + ((0.278 \text{ kip}) \times (0 \text{ ft}))}{1.57 \times (48 \text{ in})}$$

$$M_o = 0.19156 \text{ kipft/ft}$$

E - Distance from lateral load to resisting surface,

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

$$E = \frac{(0.19156 \text{ kipft/ft})}{(0.044268 \text{ kip/ft})}$$

$$E = 4.3273 \text{ ft}$$

a - Distance from resting surface to pivot point,

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

$$a = \frac{(4 \times (0.19156 \text{ kipft/ft}) \times (6.25 \text{ ft})) + (3 \times (0.044268 \text{ kip/ft}) \times (6.25 \text{ ft})^2)}{(6 \times (0.19156 \text{ kipft/ft})) + (4 \times (0.044268 \text{ kip/ft}) \times (6.25 \text{ ft}))}$$

$$a = 4.4222 \text{ ft}$$

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

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

$$V_{max} = ((0.044268 \text{ kip/ft}) \times (48 \text{ in})) \times \left[1 - \left[3 \times \left(\frac{4 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4222 \text{ ft})}{(6.25 \text{ ft})} \right)^2 \right] + \left[4 \times \left(\frac{3 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4222 \text{ ft})}{(6.25 \text{ ft})} \right)^3 \right] \right]$$

$$V_{max} = 0.33437 \text{ kip}$$

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

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

$$M_{max} = ((0.044268 \text{ kip/ft}) \times (48 \text{ in}) \times (6.25 \text{ ft})) \times \left[\left(\frac{(4.3273 \text{ ft})}{(6.25 \text{ ft})} + \frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right) - \left[\left(\frac{4 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 3 \right) \times \left(\frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^3 \right] + \left[\left(\frac{3 \times (4.3273 \text{ ft})}{(6.25 \text{ ft})} + 2 \right) \times \left(\frac{(4.4222 \text{ ft})}{2 \times (6.25 \text{ ft})} \right)^4 \right] \right]$$

$$M_{max} = 0.94573 \text{ kipft}$$

Minimum Reinforcement Check (LRFD)

Parameters:

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

Table 22.4.2.1

Longitudinal reinforcement:

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

22.4.2.2, 10.6.1.1

$A_{st,required}$

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

$$A_{st,required} = \text{Min} \left[\frac{\frac{(8.994 \text{ kip})}{(0.65) \times (0.8)} - (0.85 \times (2.5 \text{ ksi}) \times (2304 \text{ in}^2))}{(60 \text{ ksi}) - (0.85 \times (2.5 \text{ ksi}))}, (0.08 \times (2304 \text{ in}^2)) \right]$$

$$A_{st,required} = -84.297 \text{ in}^2$$

A_{min} - Governing minimum reinforcement area,

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

$$A_{min} = \text{Max} [(-84.297 \text{ in}^2), (0.0018 \times (2304 \text{ in}^2))]$$

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

n_{rebar} - Required number of reinforcement,

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

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

$$n_{rebar} = 14$$

A_{st} - Actual total reinforcement area,

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

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

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

Ratio - Capacity

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

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

$$\text{Ratio} = 0.96556$$

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

25.2.3

s_{rebar} - Minimum spacing of reinforcement,

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

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

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

Ties:

25.7.2.2

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

25.7.2.1

s_{ties} - Maximum spacing of ties,

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

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

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

Summary:

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

Ties: #3(0.375 in) - 10 in

Axial Compression Strength (ACI 318-19, LRFD)

22.4.2.2

ϕP_N - Allowable axial compressive strength

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

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

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

Ratio - Capacity

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

$$\text{Ratio} = \frac{(8.994 \text{ kip})}{(2675.2 \text{ kip})}$$

$$\text{Ratio} = 0.003362$$

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

Shear Strength (ACI 318-19, LRFD)

Parameters:

22.5.2.2

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

$$d = 0.80 D$$

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

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

22.5.5.1.3

λ_s - size effect modification factor

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

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

$$\lambda_s = 0.64282$$

22.5.5.1.1

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

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

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

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

22.5.5.1.1(a)

The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$, $P = 8.994 \text{ kip} \rightarrow 8994 \text{ lbf}$,
 $V_{c,a}$ - Shear strength of concrete (a)

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

$$V_{c,a} = \left[2 \times (0.64282) \times \sqrt{(2500 \text{ psi})} + \frac{(8994 \text{ lbf})}{6 \times (2304 \text{ in}^2)} \right] \times (48 \text{ in}) \times (38.4 \text{ in})$$

$$V_{c,a} = 119.68 \text{ kip}$$

22.5.5.1.2

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

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

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

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

V_c - Governing shear strength of concrete

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

$$V_c = \text{Min}[(296.21 \text{ kip}), (119.68 \text{ kip}), (348.89 \text{ kip})]$$

$$V_c = 119.68 \text{ kip}$$

| | | |
|-----------------|---|---|
| <p>22.5.1.2</p> | <p>The following variables were converted to be consistent with empirical formula $f'_{ck} = 2.5 \text{ ksi} \rightarrow 2500 \text{ psi}$.</p> <p>$V_{s,a}$ - Shear strength of steel (a)</p> $V_{s,a} = 8 \sqrt{f'_{ck}} b_w d$ $V_{s,a} = 8 \times \sqrt{(2500 \text{ psi})} \times (48 \text{ in}) \times (38.4 \text{ in})$ $V_{s,a} = 737.28 \text{ kip}$ <p>A_v - Ties rebar area,</p> $A_v = \frac{\pi d_{ties}^2}{4}$ $A_v = \frac{\pi \times (0.375 \text{ in})^2}{4}$ $A_v = 0.11045 \text{ in}^2$ <p>22.5.8.5.3 $V_{s,b}$ - Shear strength of steel (b)</p> $V_{s,b} = \frac{2 A_v f_{yt} d}{s_{ties}}$ $V_{s,b} = \frac{2 \times (0.11045 \text{ in}^2) \times (60 \text{ ksi}) \times (38.4 \text{ in})}{(10 \text{ in})}$ $V_{s,b} = 50.894 \text{ kip}$ <p>V_s - Governing shear strength of steel</p> $V_s = \text{MIN}[V_{s,a}, V_{s,b}]$ $V_s = \text{MIN}[(737.28 \text{ kip}), (50.894 \text{ kip})]$ $V_s = 50.894 \text{ kip}$ <p>22.5.1.1 ϕV_n - Allowable shear strength</p> $\phi V_n = \phi (V_c + V_s)$ $\phi V_n = (0.65) \times ((119.68 \text{ kip}) + (50.894 \text{ kip}))$ $\phi V_n = 110.88 \text{ kip}$ <p>Considering x-direction:</p> <p>$V_{max} = 8.2956 \text{ kip}$ - Maximum shear force in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(8.2956 \text{ kip})}{(110.88 \text{ kip})}$ $\text{Ratio} = 0.074819$ <p>Considering z-direction:</p> <p>$V_{max} = 0.33437 \text{ kip}$ - Maximum shear force in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{V_{max}}{\phi V_n}$ $\text{Ratio} = \frac{(0.33437 \text{ kip})}{(110.88 \text{ kip})}$ $\text{Ratio} = 0.0030157$ | <p>Status: PASS Ratio: 0.070</p> <p>Status: PASS Ratio: 0.000</p> |
| | <p>Flexural Strength (ACI 318-19, LRFD)</p> <p>S_m - Section modulus</p> $S_m = \frac{b D^2}{6}$ $S_m = \frac{(48 \text{ in}) \times (48 \text{ in})^2}{6}$ $S_m = 18432 \text{ in}^3$ | |

| | | |
|------------------|--|---|
| <p>14.5.2.1b</p> | <p>$\lambda = 1$ - Concrete modification factor (Normal concrete), Allowable flexural strength: M_n shall be the lesser of: $\phi M_{n,1}$</p> $\phi M_{n,1} = \phi \times 5 \times \lambda \times \sqrt{f'_c} \times S_m$ $\phi M_{n,1} = 0.65 \times 5 \times 1 \times \sqrt{2.5 \text{ ksi}} \times 18432.001 \text{ in}^3$ $\phi M_{n,1} = 249.600 \text{ kipft}$ <p>$\phi M_{n,2}$</p> $\phi M_{n,2} = \phi \times 0.85 \times f'_c \times S_m$ $\phi M_{n,2} = (0.65) \times 0.85 \times (2.5 \text{ ksi}) \times (18432 \text{ in}^3)$ $\phi M_{n,2} = 2121.6 \text{ kipft}$ <p>Therefore, ϕM_n - Allowable flexural strength,</p> $\phi M_n = \text{MIN}[\phi M_{n,1}, \phi M_{n,2}]$ $\phi M_n = \text{MIN}[(249.6 \text{ kipft}), (2121.6 \text{ kipft})]$ $\phi M_n = 249.6 \text{ kipft}$ <p>Considering x-direction: $M_{max} = 24.964 \text{ kipft}$ - Maximum moment in the x-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(24.964 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.10002$ | <p>Status: PASS Ratio: 0.100</p> |
| | <p>Considering z-direction: $M_{max} = 0.94573 \text{ kipft}$ - Maximum moment in the z-direction, Ratio - Capacity</p> $\text{Ratio} = \frac{M_{max}}{\phi M_n}$ $\text{Ratio} = \frac{(0.94573 \text{ kipft})}{(249.6 \text{ kipft})}$ $\text{Ratio} = 0.003789$ | <p>Status: PASS Ratio: 0.000</p> |