EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

GENERAL INFORMATION

Example Statement: Example 10 demonstrates a design procedure for a drilled shaft foundation for a cantilever sign structure. The cantilever supports a sign panel attached to the horizontal support. The example is only for the design of the shaft foundation. It does not discuss cover design of the members and attachment.

The design follows the LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, First Edition 2015, with 2017 updates (AASHTO LTS), with references to AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO). Example 10 was designed with a geotechnical investigation performed on the soil. If one does not have geotechnical data, it is CDOT’s preference to use the Brom’s method in Section 13 of the AASHTO LTS to determine shaft embedment.

MATERIAL PROPERTIES

Concrete: CDOT Concrete Class BZ
- Concrete Compressive Strength $f'_c = 4$ ksi
- Concrete Unit Weight $\gamma_c = 150$ pcf

Steel: Reinforcing Steel
- Grade 60 Reinforcing Steel $f_y = 60$ ksi

Steel: Steel Members
- Steel Density $\gamma_{steel} = 490$ pcf

Aluminum: Sign Panels
- Aluminum Density $\gamma_{aluminum} = 175$ pcf

SIGN STRUCTURE GEOMETRY INFORMATION (Refer to Figure 1)

- Pole Length $L_{pole} = 22.00$ ft.
- Pole Base Diameter (outside diameter, o.d.) $\phi_{pole-B} = 15.50$ in.
- Pole Top Diameter (o.d.) $\phi_{pole-T} = 12.50$ in.
- Pole Wall Thickness $t_{pole} = 0.1875$ in.
- Depth to Arm $D_{arm} = 1.50$ ft.
- Arm Length $L_{arm} = 16.00$ ft.
- Arm Base Diameter (o.d.) $\phi_{arm-B} = 10.00$ in.
- Arm End Diameter (o.d.) $\phi_{arm-E} = 6.25$ in.
- Arm Wall Thickness $t_{arm} = 0.1875$ in.
- Shaft Depth $D_{shaft} = 13.00$ ft.
- Shaft Diameter $\phi_{shaft} = 36$ in.
- Number of Sign Panels $4$
EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

SIGN PANEL GEOMETRY INFORMATION

<table>
<thead>
<tr>
<th></th>
<th>Length</th>
<th>Height</th>
<th>e_sp</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sign Panel 1</td>
<td>8.00 ft</td>
<td>6.00 ft</td>
<td>11.00 ft</td>
<td>48.00 ft²</td>
</tr>
</tbody>
</table>

1. LOAD CALCULATION

Use the load combinations and factors from AASHTO LTS T3.4-1 for all loads acting on the sign structure. Determine the loads at the top of the shaft foundation:

APPLIED LOADS

DC - dead load of structural components and nonstructural attachments
LL - live load is considered for designing members for walkways and service platforms
ICE - ice and wind on ice do not practically control and have been removed from the specifications
W - wind load is based on the pressure of the wind acting horizontally on all components

AASHTO LTS 3
EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

Dead Loads (DC)

*Weight is based on the typical weight of steel and aluminum

<table>
<thead>
<tr>
<th>Component</th>
<th>Weight DC (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pole Weight</td>
<td>DC1 = 0.61</td>
</tr>
<tr>
<td>Arm Weight</td>
<td>DC2 = 0.25</td>
</tr>
<tr>
<td>Sign Weight</td>
<td>DC3 = 0.15</td>
</tr>
</tbody>
</table>
| Misc. Weight (Anchors and Support)| DC4 = 0.08       | *Assumed 7/32" Sign Thickness
|                                  |                  | *Assumed to be 50% of Sign Weight

Live Loads (LL)

Is LL applicable? no

Ice Loads (ICE)

Is ICE applicable? no

Wind Loads (W)

Mean Recurrence Interval

<table>
<thead>
<tr>
<th>Basic Wind Speed</th>
<th>V = 120.00 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height and Exposure Factor for Signs and Arm</td>
<td>Kx = 0.90</td>
</tr>
<tr>
<td>Height and Exposure Factor for Pole</td>
<td>Ky = 0.86</td>
</tr>
<tr>
<td>Directionality Factor</td>
<td>Kd = 0.85</td>
</tr>
<tr>
<td>Gust Effect Factor</td>
<td>G = 1.14</td>
</tr>
<tr>
<td>Velocity Conversion Factor - Ext Event</td>
<td>C_{V,Ext} = 0.80</td>
</tr>
<tr>
<td>Drag Coefficient for Members</td>
<td>Cd-members = 0.45</td>
</tr>
<tr>
<td>Drag Coefficient for Sign Panels</td>
<td>C_{d,sp} = 1.19</td>
</tr>
</tbody>
</table>

Wind Pressure on Members

\[ P_d = 0.00256 K_x K_d G V^2 C_d = 14.50 \text{ psf} \]

Wind Pressure on Sign Panels

\[ P_d = 0.00256 K_x K_d G V^2 C_d = 38.35 \text{ psf} \]

Pole Surface Area (along x axis)

\[ A_{1x} = 25.67 \text{ ft}^2 \]

Pole Surface Area (along z axis)

\[ A_{1z} = 25.67 \text{ ft}^2 \]

Arm Surface Area (along x axis)

\[ A_{2x} = 10.83 \text{ ft}^2 \]

Sign Panels Surface Area (along x axis)

\[ A_{3x} = 48.00 \text{ ft}^2 \]

Wind Load (x-direction)

\[ W = \sum A \times P_d = W_x = 0.37 \text{ kip} = A_{1x} \times P_{z-members} \]

Wind Load on Signs (z-direction)

\[ W_{z-sign} = 1.84 \text{ kip} = A_{3x} \times P_{z-sign panels} \]

Wind Load on Arm (z-direction)

\[ W_{z-arm} = 0.16 \text{ kip} = A_{2x} \times P_{z-members} \]

Wind Load on Pole (z-direction)

\[ W_{z-pole} = 0.37 \text{ kip} = A_{1x} \times P_{z-members} \]
UNFACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

<table>
<thead>
<tr>
<th>Load</th>
<th>Description</th>
<th>Load Direction (x,y,z)</th>
<th>Load (kip)</th>
<th>Moment Arm (ft.)</th>
<th>Moment Direction (x,y,z)</th>
<th>Moment at the Top of the Caisson (kip-ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC₁</td>
<td>Pole Weight</td>
<td>Y</td>
<td>0.61</td>
<td>0.00</td>
<td>Z</td>
<td>0.00</td>
</tr>
<tr>
<td>DC₂</td>
<td>Arm Weight</td>
<td>Y</td>
<td>0.25</td>
<td>4.31</td>
<td>Z</td>
<td>1.10</td>
</tr>
<tr>
<td>DC₃</td>
<td>Sign Weight</td>
<td>Y</td>
<td>0.15</td>
<td>11.00</td>
<td>Z</td>
<td>1.68</td>
</tr>
<tr>
<td>DC₄</td>
<td>Misc. Weight</td>
<td>Y</td>
<td>0.08</td>
<td>11.00</td>
<td>Z</td>
<td>0.84</td>
</tr>
<tr>
<td>LL</td>
<td>Live Load</td>
<td>Y</td>
<td>0.00</td>
<td>0.00</td>
<td>Z</td>
<td>0.00</td>
</tr>
<tr>
<td>Wₓ-pole</td>
<td>Wind on Pole</td>
<td>X</td>
<td>0.37</td>
<td>6.73</td>
<td>Z</td>
<td>2.51</td>
</tr>
<tr>
<td>Wᵧ-sign/arm</td>
<td>Wind on Signs &amp; Arm</td>
<td>Z</td>
<td>2.00</td>
<td>20.50</td>
<td>X</td>
<td>40.95</td>
</tr>
<tr>
<td>Wᵧ-sign</td>
<td>Wind on Signs</td>
<td>Z</td>
<td>1.84</td>
<td>11.00</td>
<td>Y</td>
<td>20.25</td>
</tr>
<tr>
<td>Wᵧ-arm</td>
<td>Wind on Arm</td>
<td>Z</td>
<td>0.16</td>
<td>4.31</td>
<td>Y</td>
<td>0.68</td>
</tr>
<tr>
<td>Wᵧ-pole</td>
<td>Wind on Pole</td>
<td>Z</td>
<td>0.37</td>
<td>11.00</td>
<td>X</td>
<td>4.09</td>
</tr>
</tbody>
</table>

LOAD COMBINATIONS

AASHTO LTS T3.4-1

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>γ&lt;sub&gt;DC&lt;/sub&gt;</th>
<th>γ&lt;sub&gt;LL&lt;/sub&gt;</th>
<th>γ&lt;sub&gt;W&lt;/sub&gt;</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I</td>
<td>1.25</td>
<td>1.60</td>
<td>-</td>
<td>Gravity</td>
</tr>
<tr>
<td>Extreme Ia</td>
<td>1.10</td>
<td>-</td>
<td>1.00</td>
<td>Wind max</td>
</tr>
<tr>
<td>Extreme Ib</td>
<td>0.90</td>
<td>-</td>
<td>1.00</td>
<td>Wind min</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>-</td>
<td>1.00</td>
<td>Translation</td>
</tr>
</tbody>
</table>

SUMMARY OF FACTORED LOADS AND MOMENTS AT TOP OF SHAFT

Moments taken about the centerline of the shaft

\[ U = γ_{DC}DC + γ_{LL}LL + γ_{W}W \]

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial (kip)</th>
<th>Moment about x-axis (kip-ft.)</th>
<th>Moment about y-axis* (kip-ft.)</th>
<th>Moment about z-axis (kip-ft.)</th>
<th>Shear in the x-axis (kip)</th>
<th>Shear in the z-axis (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength I</td>
<td>1.37</td>
<td></td>
<td></td>
<td>4.53</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Extreme Ia</td>
<td>1.20</td>
<td>45.05</td>
<td>20.92</td>
<td>6.49</td>
<td>0.37</td>
<td>2.37</td>
</tr>
<tr>
<td>Extreme Ib</td>
<td>0.98</td>
<td>45.05</td>
<td>20.92</td>
<td>5.77</td>
<td>0.37</td>
<td>2.37</td>
</tr>
<tr>
<td>Service I</td>
<td>1.09</td>
<td>45.05</td>
<td>20.92</td>
<td>6.13</td>
<td>0.37</td>
<td>2.37</td>
</tr>
</tbody>
</table>

*Μ<sub>e</sub> to be used for torsion calculation
2. SHAFT CAPACITY

Run static L-PILE analysis with parameters from geotechnical report and calculated factored loads.

L-PILE INPUT

Soil Properties

*From Geotechnical Report

| Top of Boring Elevation | El_boring_top = 5297.00 |
| Bottom of Boring Elevation | El_boring_bot = 5270.00 |
| Top of Shaft Elevation | El_caisson_top = 5297.50 |
| Bottom of Shaft Elevation | El_caisson_bot = 5284.50 |

<table>
<thead>
<tr>
<th>Top of Soil Elev.</th>
<th>Soil Type</th>
<th>Unit Weight (pcf)</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (psf)</th>
<th>$\varepsilon_{50}$</th>
<th>k (pci)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5297.00</td>
<td>Stiff Clay w/o free water using k</td>
<td>120.00</td>
<td>0.00</td>
<td>2000.00</td>
<td>0.006</td>
<td>500.00</td>
</tr>
<tr>
<td>5290.00</td>
<td>Stiff Clay w/o free water using k</td>
<td>130.00</td>
<td>0.00</td>
<td>2500.00</td>
<td>0.005</td>
<td>1000.00</td>
</tr>
</tbody>
</table>

Shaft Section Properties

<table>
<thead>
<tr>
<th>Section</th>
<th>Round Concrete Shaft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length of Section</td>
<td>$D_{shaft} = 13.00$ ft.</td>
</tr>
<tr>
<td>Length of Section in Bedrock</td>
<td>$D_{rock} = 5.50$ ft.</td>
</tr>
<tr>
<td>Section Diameter</td>
<td>$\phi_{shaft} = 36$ in.</td>
</tr>
<tr>
<td>Longitudinal Rebar Size</td>
<td># 8</td>
</tr>
<tr>
<td>Longitudinal Rebar Count</td>
<td>13</td>
</tr>
<tr>
<td>Concrete Cover to Inside Edge of Stirrup Bar</td>
<td>3.625 in.</td>
</tr>
<tr>
<td>Stirrup Size</td>
<td># 5</td>
</tr>
<tr>
<td>Stirrup Spacing</td>
<td>12 in.</td>
</tr>
</tbody>
</table>

INPUT LOADS

L-Pile models in only one plane, therefore:

Shear in the X Direction is paired with Moment in the Z Direction
Shear in the Z Direction is paired with Moment in the X Direction

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Pile-Head Loading Condition</th>
<th>Shear (lb)</th>
<th>Moment (lb-in)</th>
<th>Axial (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>0</td>
<td>54,347</td>
<td>1,367</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>372</td>
<td>77,892</td>
<td>1,203</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>2,370</td>
<td>540,557</td>
<td>1,203</td>
</tr>
<tr>
<td>4</td>
<td>1</td>
<td>372</td>
<td>69,196</td>
<td>984</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>2,370</td>
<td>540,557</td>
<td>984</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>372</td>
<td>73,544</td>
<td>1,093</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>2,370</td>
<td>540,557</td>
<td>1,093</td>
</tr>
</tbody>
</table>
**L-PILE OUTPUT**

*Agg size assumed to be 0.75”*

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>13 #8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear Distance Between Bars</td>
<td>5.64 in.</td>
</tr>
<tr>
<td>Spacing Check for Min Spacing</td>
<td>&gt;</td>
</tr>
<tr>
<td>Min Clear Allowed, Max(1.5d, 1.5*Agg Size, 1.5&quot;)</td>
<td>1.50 in.</td>
</tr>
<tr>
<td>Min Clear Allowed, Max(5*Agg Size, 5&quot;)</td>
<td>5.00 in.</td>
</tr>
<tr>
<td>Area of Steel</td>
<td>10.27 in.$^2$</td>
</tr>
<tr>
<td>Percentage of Steel</td>
<td>1.01%</td>
</tr>
<tr>
<td></td>
<td>&gt;</td>
</tr>
<tr>
<td></td>
<td>0.80%</td>
</tr>
</tbody>
</table>

| Maximum Pile-Head Deflection | 0.0043 in. |
| Maximum Shear Force | 7,261 lbs |
| Maximum Bending Moment | 567,170 lb-in |
| Axial Thrust at Max Moment Case | 1,203 lbs |

**Lateral Pile Deflection (in.) vs Depth (ft.)**

The maximum deflection, at the top of the caisson is 0.0043”, which is considered zero; therefore, the shaft is deemed stable for the length used per the Engineer's judgment.
Bending Moment (in-kip) vs Depth (ft.)

The maximum factored moment is less than the maximum resistance moment. The shaft is considered stable per the reinforcement and size.
Shear Force (kips) vs Depth (ft.)
AXIAL RESISTANCE

Unit End Bearing Resistance \( q_p = 18.00 \) ksf Geotechnical Report
Unit Side Resistance \( q_s = 1.00 \) ksf Geotechnical Report
End Bearing Factor \( \phi_{qp} = 0.40 \) Geotechnical Report
Side Resistance Factor \( \phi_{qs} = 0.45 \) Geotechnical Report
Shaft End Bearing Area \( A_{shaft} = \pi d^2/4 = 7.07 \) ft.\(^2\)
Shaft Perimeter \( P_{shaft} = \pi d = 9.42 \) ft.
Depth in Bedrock \( D_{rock} = 5.50 \) ft.
End Bearing Resistance \( \phi_{qp} q_p A_{shaft} = 50.89 \) kip AASHTO Eq. 10.8.3.5-2
Side Shear Resistance \( \phi_{qs} q_s P_{shaft} D_{rock} = 23.33 \) kip AASHTO Eq. 10.8.3.5-3
Ultimate Shaft Resistance \( R_u = \phi R_n = \phi_{qp} R_p + \phi_{qs} R_s = 74.22 \) kip AASHTO Eq. 10.8.3.5-1

BENDING RESISTANCE

L-Pile provides Nominal Moment Resistance for each axial value.

The maximum factored applied moment from each L-Pile case with varying axial is compared to the nominal moment resistance provided by L-Pile.

\[
\phi M_n = M_u \geq M_{applied} \\
\phi = 0.75 \\
AASHTO 5.5.4.2
\]

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Axial (lb)</th>
<th>Nominal Moment Resistance, ( M_n ) (kip-in.)</th>
<th>( \phi )</th>
<th>Ultimate Moment Resistance, ( M_u ) (kip-in.)</th>
<th>Factored Applied Moment, ( M_{applied} ) (kip-in.)</th>
<th>Check</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>984</td>
<td>8,472.87</td>
<td>0.75</td>
<td>6,354.65</td>
<td>540.56</td>
<td>OK!</td>
</tr>
<tr>
<td>7</td>
<td>1,093</td>
<td>8,474.12</td>
<td>0.75</td>
<td>6,355.59</td>
<td>540.56</td>
<td>OK!</td>
</tr>
<tr>
<td>3</td>
<td>1,203</td>
<td>8,475.37</td>
<td>0.75</td>
<td>6,356.53</td>
<td>540.56</td>
<td>OK!</td>
</tr>
<tr>
<td>1</td>
<td>1,367</td>
<td>8,477.25</td>
<td>0.75</td>
<td>6,357.94</td>
<td>54.35</td>
<td>OK!</td>
</tr>
</tbody>
</table>
**SHEAR AND TORSION RESISTANCE**

*The side shear resistance of soil for torsion effects is checked at the end of this example.

**Shear Force**
\[ V_u = 7.26 \text{ kip} \]

**Torsion**
\[ M_y = T_u = 20.92 \text{ k-ft.} \]

**Flexure**
\[ M_u = 45.05 \text{ k-ft.} \]

**Tension**
\[ N_u = 15.15 \text{ kip} \]

**Phi for Shear and Torsion**
\[ \phi = 0.90 \quad \text{AASHTO 5.5.4.2} \]

**Concrete Cover to Reinforcing & Bar Size:**
- **Side Cover**
  \[ \text{clr} = 3.00 \text{ in.} \]
- **Stirrup Bar Diameter**
  \[ d_{\text{stirrup}} = 0.63 \text{ in.} \]

**Nominal Resistance**
\[ M_n = 706.07 \text{ k-ft.} \quad \text{L-Pile Output} \]

**Area of Flexural Reinforcement**
\[ A_f = 5.14 \text{ in.}^2 \quad \text{Half of the reinforcement in shaft} \]

**Dia of Circle Passing Through Long. Reinf**
\[ D_r = 27.75 \text{ in.}^3 \]

**Depth to Flexural Reinforcement**
\[ d_s = 26.83 \text{ in.} = \frac{D_{\text{shaft}}}{2} + \frac{D_r}{\pi} \]

**Torsional Cracking Moment**

**Area of Concrete Perimeter**
\[ A_{cp} = 1,018 \text{ in.}^2 \]

**Concrete Perimeter**
\[ p_c = 113.10 \text{ in.} \]

**Compressive Stress at Centroid of Section**
\[ f_{pc} = 0.00 \text{ ksi} \]

**Torsional Cracking Moment**
\[ T_{cr} = 2,308.54 \text{ k-in.} \]

\[ 0.25\phi T_{cr} = 519.42 \text{ k-in.} \]

\[ T_u = 251.08 \text{ k-in.} \quad \text{AASHTO 5.7.2.1-3} \]

Torsional effects can be neglected

**Design Factored Shear Force**
\[ V_u = 7.26 \text{ kip} \]

**Shear Stress on Concrete**

**Effective Shear Depth**
\[ d_v = \text{max of} \begin{cases} M_n \\ A_s f_y \\ 0.9 * d_s \\ 0.72 * h \end{cases} \]

\[ \frac{M_n}{A_s f_y} = 27.50 \text{ in.} \quad \text{Maximum} \]
\[ 0.9 * d_s = 24.15 \text{ in.} \]
\[ 0.72 * h = 25.92 \text{ in.} \]

\[ d_v = 27.50 \text{ in.} \]
Shear Stress  
\[ v_u = \frac{V_u}{\phi V_c} = v_u = 0.0081 \text{ ksi} \]  
AASHTO Eq. 5.7.2.8-1

Transverse Reinforcement

Transverse Reinforcement is required where: 
\[ V_u > 0.5 \phi V_c \]  
AASHTO Eq. 5.7.2.3-1

\[ V_u = 7.26 \text{ kip} \]

\[ 0.5 \phi V_c = 115.15 \text{ kip} \]

Transverse reinforcement not necessary

Minimum Transverse Reinforcement  
\[ A_{v, min} \geq 0.03166 \sqrt{\frac{f'_c b_v s}{F_y}} \]  
AASHTO Eq. 5.7.2.5-1

\[ A_{v, min} \geq 0.46 \text{ in.}^2 \]

\[ A_{v, prov'd} = 0.62 \text{ in.}^2 \]

OK!

Maximum Spacing of Transverse Reinforcement  
AASHTO 5.7.2.6

\[ v_u = 0.008 \text{ ksi} \]

\[ 0.125 f'_c = 0.500 \text{ ksi} \]

If \( v_u < 0.125 f'_c \), then:  
\[ s_{max} = 0.8 d_v \leq 24.0 \]  
AASHTO Eq. 5.7.2.6-1

If \( v_u \geq 0.125 f'_c \), then:  
\[ s_{max} = 0.4 d_v \leq 12.0 \]  
AASHTO Eq. 5.7.2.6-2

\[ s_{max} = 22.00 \text{ in.} \]

\[ s_{v, prov'd} = 12.00 \text{ in.}^2 \]

OK!

Maximum Nominal Shear Resistance  
AASHTO 5.7.3.3

Nominal Shear Resistance  
\[ 0.25 f'_c b_v d_v \leq V_n = 990.01 \text{ kip} \]  
AASHTO Eq. 5.7.3.3-2

\[ \phi V_n = 891.01 \text{ kip} \]

\[ V_u = 7.26 \text{ kip} \]

OK!

\[ \varepsilon_s = \frac{(|M_u|/d_v + 0.5 N_u + |V_u|)}{E_s A_s} \]  
AASHTO Eq. 5.7.3.4.2-4

Net Longitudinal Tensile Strain  
\[ \varepsilon_s = 0.0002 \]
EXAMPLE 10 - SIGN STRUCTURE FOUNDATION DESIGN

For sections containing at least the minimum amount of transverse reinforcement specified in Art. 5.7.2.5, the value of $\beta$ may be determined by the following equation:

$$\beta = \frac{4.8}{(1 + 750\varepsilon_s)} \quad \text{AASHTO Eq. 5.7.3.4.2-1}$$

$$\beta = 4.09$$

$$\varepsilon_s = 29.81 \quad \text{AASHTO Eq. 5.7.3.4.2-3}$$

Nominal Shear Resistance of Concrete

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}b_v d_v \quad \text{AASHTO Eq. 5.7.3.3-3}$$

$V_c = 255.88$ kip

$> 7.26$ kip

OK!

Side Shear Resistance of Soil in Torsion

Per CDOT’s experience, the soil torsion capacity may control the shaft length. If the drilled shaft sees torsion, the following applicable checks should be completed. Refer to Report No. CDOT-DTD-R-2004-8 for equations and procedure.

Cohesive Soil Resistance

It is CDOT’s approach that the soil resistance to torsion in cohesive soils is based on the drilled shaft embedment area into the soil, neglecting the top 1.5’ of section length. Perform the following check if the drilled shaft is in cohesive soil.

<table>
<thead>
<tr>
<th>Torsion</th>
<th>$T_u = 20.92$ k-ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil profile used for example</td>
<td>cohesion, $s_u = 2000$ psf</td>
</tr>
<tr>
<td>Assumed Phi for Torsion, per SF = 1.25</td>
<td>$\phi = 0.80$</td>
</tr>
<tr>
<td>Section Diameter</td>
<td>$\phi_{shaft} = 36$ in</td>
</tr>
<tr>
<td>Length of Section</td>
<td>$D_{shaft} = 13.00$ ft.</td>
</tr>
</tbody>
</table>

Drilled shaft side resistance

$$T_s = \frac{\pi \cdot \phi_{shaft}^2}{2} (D_{shaft} - 1.5\phi_{shaft}) \cdot s_u$$

$$T_s = 240.33$ k-ft. |

Drilled shaft toe resistance

$$T_t = \frac{\pi \cdot \phi_{shaft}^3}{12} s_u$$

$$T_t = 14.14$ k-ft. |

Nominal Total Torsion Resistance

$$T_n = (T_s + T_t) = 254.47$ k-ft. |

$$\phi T_n = 203.58$ k-ft. |

$> 20.92$ k-ft.

OK!
Cohesionless Soil Resistance

It is CDOT’s approach that the soil resistance to torsion in cohesionless soils is based on the drilled shaft embedment into the soil. Perform the following check if the drilled shaft is in cohesionless soil.

- **Torsion**  
  \( T_u = 20.92 \text{ k-ft.} \)

- **Soil profile used for example**  
  Unit weight, \( \gamma = 120 \text{ pcf} \)

- **Soil profile used for example**  
  Friction angle, \( \varphi = 30.00 \text{ degrees} \)

- **Assumed Phi for Torsion, per SF = 1.25**  
  \( \phi = 0.80 \)

- **Section Diameter**  
  \( \phi_{\text{shaft}} = 36.00 \text{ in} \)

- **Length of Section**  
  \( D_{\text{ shaft}} = 13.00 \text{ ft.} \)

- **Weight of Section**  
  \( W = 13.78 \text{ kip} \)

\[
T_s = \frac{\pi \cdot \phi_{\text{shaft}}^2}{2} D_{\text{ shaft}} \cdot r_s
\]

\[
T_t = \frac{\phi_{\text{ shaft}}}{3} W \cdot \tan \varphi
\]

\[
K = \frac{2D_{\text{ shaft}}}{3\phi_{\text{ shaft}}} (1 - \sin \varphi) = 1.44
\]

\[
r_s = Ky \frac{D_{\text{ shaft}}}{2} \tan \varphi = 0.65 \text{ ksf}
\]

\[
T_s = 119.55 \text{ k-ft.}
\]

\[
T_t = 7.96 \text{ k-ft.}
\]

\[
T_n = (T_s + T_t) = 127.51 \text{ k-ft.}
\]

\[
\phi T_n = 102.00 \text{ k-ft.}
\]

\[
T_u = 20.92 \text{ k-ft.}
\]

\[
> \text{ OK!}
\]