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 $γ_c = 150$ pcf

Steel: Reinforcing Steel

- Grade 60 Reinforcing Steel $f_y = 60$ ksi

Steel: Steel Members

- Steel Density $γ_{steel} = 490$ pcf

Aluminum: Sign Panels

- Aluminum Density $γ_{aluminum} = 175$ pcf

**SIGN STRUCTURE GEOMETRY INFORMATION** (Refer to Figure 1)

- Pole Length $L_{pole} = 22.00$ ft.
- Pole Base Diameter (outside diameter, o.d.) $d_{pole-B} = 15.50$ in.
- Pole Top Diameter (o.d.) $d_{pole-T} = 12.50$ 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.) $d_{arm-B} = 10.00$ in.
- Arm End Diameter (o.d.) $d_{arm-E} = 6.25$ in.
- Arm Wall Thickness $t_{arm} = 0.1875$ in.
- Shaft Depth $D_{shaft} = 13.00$ ft.
- Shaft Diameter $d_{shaft} = 36$ in.
- Number of Sign Panels $n = 1$
SIGN PANEL GEOMETRY INFORMATION

<table>
<thead>
<tr>
<th>Sign Panel</th>
<th>Length</th>
<th>Height</th>
<th>e_sp</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.00 ft.</td>
<td>6.00 ft.</td>
<td>11.00 ft.</td>
<td>48.00 ft.²</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

(Other loads not listed here may be applicable for different design cases.)

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
Dead Loads (DC)  
*Weight is based on the typical weight of steel and aluminum

- Pole Weight: \( DC_1 = 0.61 \) kip  
- Arm Weight: \( DC_2 = 0.25 \) kip  
- Sign Weight: \( DC_3 = 0.15 \) kip *Assumed 7/32” Sign Thickness  
- Misc. Weight (Anchors and Sign Support): \( DC_4 = 0.08 \) kip *Assumed to be 50% of Sign Weight

Live Loads (LL)  
AASHTO LTS 3.6

- Is LL applicable? no

Ice Loads (ICE)  
AASHTO LTS 3.7

- Is ICE applicable? no

Wind Loads (W)  
AASHTO LTS 3.8

- Mean Recurrence Interval: \( MRI = 1700 \) BDM 32.3.1.3
- Basic Wind Speed: \( V = 120.00 \) mph BDM 32.3.1.3
- Height and Exposure Factor for Signs and Arm: \( K_x = 0.90 \) AASHTO LTS Eq. 3.8.4-1
- Height and Exposure Factor for Pole: \( K_z = 0.86 \) AASHTO LTS Eq. 3.8.4-1
- Directionality Factor: \( K_d = 0.85 \) AASHTO LTS 3.8.5
- Gust Effect Factor: \( G = 1.14 \) AASHTO LTS 3.8.6
- Velocity Conversion Factor - Ext Event: \( C_v \cdot V \cdot d = C_v \cdot V \cdot \phi_{pole-avg} = 112.00 \) AASHTO LTS 3.8.7
- Velocity Conversion Factor: \( C_v = 1.00 \) AASHTO LTS 3.8.7
- Drag Coefficient for Members: \( C_{d-members} = 0.45 \) AASHTO LTS 3.8.7
- Drag Coefficient for Sign Panels: \( C_{d-sps} = 1.19 \) *rounded up AASHTO LTS 3.8.7

Wind Pressure on Members

\[
P_z = 0.00256 \cdot K_z \cdot K_d \cdot G^2 \cdot C_d = 14.50 \text{ psf} \quad \text{AASHTO LTS Eq. 3.8.1-1}
\]

Wind Pressure on Sign Panels

\[
P_z = 0.00256 \cdot K_z \cdot K_d \cdot G^2 \cdot C_d = 38.35 \text{ psf} \quad \text{AASHTO LTS Eq. 3.8.1-1}
\]

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): \( W = \Sigma A \cdot P_z = W_x = 0.37 \text{ kip} = A_{1z} \cdot P_z \cdot \text{members} \)
- Wind Load on Signs (z-direction): \( W_{z-sign} = 1.84 \text{ kip} = A_{3x} \cdot P_z \cdot \text{sign panels} \)
- Wind Load on Arm (z-direction): \( W_{z-arm} = 0.16 \text{ kip} = A_{2x} \cdot P_z \cdot \text{members} \)
- Wind Load on Pole (z-direction): \( W_{z-pole} = 0.37 \text{ kip} = A_{1x} \cdot P_z \cdot \text{members} \)
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>Wz-sign &amp; 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>Wz-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>Wz-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>Wz-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

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( \gamma_{DC} )</th>
<th>( \gamma_{LL} )</th>
<th>( \gamma_{W} )</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

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

<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>-</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>

*M\(_{t}\) 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

<table>
<thead>
<tr>
<th>Top of Boring Elevation</th>
<th>El_{boring, top} = 5297.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom of Boring Elevation</td>
<td>El_{boring, bot} = 5270.00</td>
</tr>
<tr>
<td>Top of Shaft Elevation</td>
<td>El_{caisson, top} = 5297.50</td>
</tr>
<tr>
<td>Bottom of Shaft Elevation</td>
<td>El_{caisson, bot} = 5284.50</td>
</tr>
</tbody>
</table>

<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

- **Reinforcement**: 13 #8
- **Clear Distance Between Bars**: 5.64 in.
- **Spacing Check for Min Spacing**: >
- **Min Clear Allowed, Max(1.5\(d_b\), 1.5"Agg Size, 1.5")**: 1.50 in. AASHTO 5.10.3.1.1
- **Min Clear Allowed, Max(5"Agg Size, 5")**: 5.00 in. AASHTO 5.13.4.5.2
- **Area of Steel**: 10.27 in.\(^2\)
- **Percentage of Steel**: 1.01%

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

Applied Vertical Load \( 15.15 \text{ kip} \)

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.

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

<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 is not considered in 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

\[ cl_r = 3.00 \text{ in.} \]

Stirrup Bar Diameter

\[ d_{stirrup} = 0.63 \text{ in.} \]

Nominal Resistance

\[ M_n = 706.07 \text{ k-ft.} \]

Area of Flexural Reinforcement

\[ A_f = 5.14 \text{ in.}^2 \]

Half of the reinforcement in shaft

Dia of Circle Passing Through Long. Reinf

\[ D_r = 27.75 \text{ in.}^3 \]

Depth to Flexural Reinforcement

\[ d_s = 26.83 \text{ in.} = \frac{D_{shaft}}{2} + \frac{D_r}{\pi} \]

Torsional Cracking Moment

\[ T_{cr} = 2,290.22 \text{ k-in.} \quad \text{AASHTO Eq.5.8.2.1-4} \]

\[ 0.25\phi T_{cr} = 515.30 \text{ k-in.} \quad \text{AASHTO Eq.5.8.2.1-3} \]

Torsional effects can be neglected

Design Factored Shear Force

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

Shear Stress on Concrete

\[ A_{AS/\phi f_y} \]

Effective Shear Depth

\[ d_v = \max \left\{ \frac{M_n}{A_s f_y}, \frac{0.9 \cdot d_s}{0.72 \cdot h} \right\} \]

\[ \frac{M_n}{A_s f_y} = 27.50 \text{ in.} \quad \text{Maximum} \]

\[ 0.9 \cdot d_s = 24.15 \text{ in.} \]

\[ 0.72 \cdot h = 25.92 \text{ in.} \]

\[ d_v = 27.50 \text{ in.} \]

Shear Stress

\[ v_u = \frac{|V_u|}{\phi_b d_v} = 0.0081 \text{ ksi} \quad \text{AASHTO Eq. 5.8.2.9-1} \]
**Transverse Reinforcement**

Transverse Reinforcement is required where: \( V_u > 0.5 \phi V_c \)  
\[
V_u = \frac{7.26}{\text{kip}} \quad \text{<}
\]
\[
0.5 \phi V_c = \frac{115.15}{\text{kip}}
\]

Transverse reinforcement not necessary

**Minimum Transverse Reinforcement**

\[
A_{v, \text{min}} \geq 0.0316 \lambda \sqrt{f_{c}^{'} b_{v} s}
\]
\[
A_{v, \text{min}} \geq \frac{0.46}{\text{in.}^2}
\]
\[
A_{v, \text{prov'd}} = 0.62 \text{ in.}^2
\]

**Maximum Spacing of Transverse Reinforcement**

\[
u_u = \frac{0.008}{\text{ksi}}
\]
\[
0.125 f_{c}^{'} = \frac{0.500}{\text{ksi}}
\]

If \( \nu_u < 0.125 f_{c}^{'} \), then: 
\[
s_{\text{max}} = 0.8 d_v \leq 24.0
\]

If \( \nu_u \geq 0.125 f_{c}^{'} \), then: 
\[
s_{\text{max}} = 0.4 d_v \leq 12.0
\]
\[
s_{\text{max}} = \frac{22.00}{\text{in.}}
\]
\[
s_{v, \text{prov'd}} = \frac{12.00}{\text{in.}^2}
\]

**Maximum Nominal Shear Resistance**

\[
V_n = 0.25 f_{c}^{'} b_{v} d_v = V_n = \frac{990.01}{\text{kip}}
\]
\[
\phi V_{n} = \frac{891.01}{\text{kip}}
\]
\[
V_u = \frac{7.26}{\text{kip}}
\]

\[
\epsilon_s = \frac{(|M_u| + 0.5 N_u + |V_u|)}{E_s A_s}
\]

**Net Longitudinal Tensile Strain**

\[
\epsilon_s = 0.0002
\]
For sections containing at least the minimum amount of transverse reinforcement specified in Art. 5.8.2.5, the value of $\beta$ may be determined by the following equation:

$$\beta = \frac{4.8}{1 + 750\varepsilon_s}$$

$\beta = 4.09$

$\theta = 29.81$

Nominal Shear Resistance of Concrete

$$V_c = 0.0316\beta\lambda \sqrt{f_c' b_y d_y}$$

$V_c = 255.88 \text{ kip}$

$V_u = 7.26 \text{ kip}$

OK!