GENERAL INFORMATION

Example 11 demonstrates design procedures for cast-in-place cantilever retaining walls supported on spread footing in conformance with AASHTO and Section 11.5 of this BDM. Horizontal earth pressure is applied based on the Coulomb earth pressure theory.

Example Statement: The retaining wall supports 15'-0" of level roadway embankment measured from top of wall to top of footing. The wall will be built adjacent to the roadway shoulder where traffic is 2 ft. from the barrier face. The wall stem is 1'-6" wide to accommodate mounting a Type 7 Bridge Rail to the top of wall. See Figure 3.

Starting Element Size Assumptions:
Total Footing Width = 70% to 75% of the design height
Footing Thickness = 10% of the design height
Toe Width = 10% of design height

MATERIAL PROPERTIES

Soil: CDOT Class 1 Backfill-Drained

Footing bears on soil
Soil unit weight $\gamma_s = 0.130$ kcf
Angle of internal friction (backfill) $\varphi = 34$ deg
Wall-backfill friction angle $\delta = 2/3\varphi = 22.67$ deg
Coefficient of active earth pressure $K_a = 0.261$ (Coulomb) AASHTO Eq. 3.11.5.3-1
Coefficient of passive earth pressure $K_p = 7.60$ AASHTO Fig. 3.11.5.4-1
Active equivalent fluid weight $EFW (a) = K_a \gamma_s = 0.036$ kcf (36 pcf min) BDM 11.5
Passive equivalent fluid weight $EFW (p) = K_p \gamma_s = 0.988$ kcf

Subgrade: for bearing and sliding
Nominal design values are typically provided in the project-specific geotechnical report.
Nominal soil bearing resistance $q_n = 7.50$ ksf
Angle of internal friction (subgrade) $\varphi_{sub} = 20$ deg (for sliding)
Wall-subgrade friction angle $\delta_{sub} = 2/3\varphi_{sub} = 13.33$ deg (for shear key design)
Nominal soil sliding coefficient $\mu = \tan \varphi_{sub} = 0.36$ AASHTO C.10.6.3.4

Concrete: CDOT Concrete Class D

Concrete compressive strength $f'_c = 4.50$ ksi
Concrete unit weight $\gamma_c = 0.150$ kcf

Bridge Rail Type 7

Type 7 bridge rail weight $w_{rail} = 0.486$ klf
Center of gravity from wall back face $X_{C.G.} = 6.84$ in. (see Bridge Worksheet B-606-7A)
RESISTANCE FACTORS
When not provided in the project-specific geotechnical report, refer to the indicated AASHTO sections.

<table>
<thead>
<tr>
<th></th>
<th>$\phi$</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bearing</td>
<td>0.55</td>
<td>AASHTO T.11.5.7-1</td>
</tr>
<tr>
<td>Sliding (concrete on soil)</td>
<td>1.00</td>
<td>AASHTO T.11.5.7-1</td>
</tr>
<tr>
<td>Sliding (soil on soil)</td>
<td>1.00</td>
<td>AASHTO T.11.5.7-1</td>
</tr>
<tr>
<td>Passive pressure</td>
<td>0.50</td>
<td>AASHTO T.10.5.5.2.2-1</td>
</tr>
<tr>
<td>Extreme event</td>
<td>1.00</td>
<td>AASHTO 11.5.8</td>
</tr>
</tbody>
</table>

WALL GEOMETRY INFORMATION
See Figure 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stem Height H</td>
<td>15.00 ft.</td>
</tr>
<tr>
<td>Top of Wall Thickness T&lt;sub&gt;Top&lt;/sub&gt;</td>
<td>1.50 ft.</td>
</tr>
<tr>
<td>Bottom of Wall Thickness T&lt;sub&gt;Bot&lt;/sub&gt;</td>
<td>1.75 ft.</td>
</tr>
<tr>
<td>Width of footing B</td>
<td>10.00 ft.</td>
</tr>
<tr>
<td>Thickness of Footing T&lt;sub&gt;F&lt;/sub&gt;</td>
<td>1.25 ft.</td>
</tr>
<tr>
<td>Toe Distance S</td>
<td>2.75 ft.</td>
</tr>
<tr>
<td>Height of fill over the toe H&lt;sub&gt;TF&lt;/sub&gt;</td>
<td>2.00 ft.</td>
</tr>
<tr>
<td>Minimum Footing embedment ≥ 3 ft.</td>
<td>H&lt;sub&gt;TF&lt;/sub&gt; + T&lt;sub&gt;F&lt;/sub&gt; = 3.25 ft.</td>
</tr>
<tr>
<td>Bridge Rail Type 7 Height H&lt;sub&gt;BR&lt;/sub&gt;</td>
<td>2.92 ft.</td>
</tr>
<tr>
<td>Wall Backface to vertical surcharge R</td>
<td>2.00 ft.</td>
</tr>
<tr>
<td>Live Load Surcharge height h&lt;sub&gt;Sur&lt;/sub&gt;</td>
<td>2.00 ft.</td>
</tr>
<tr>
<td>Vehicle Collision Load (TL-4) P&lt;sub&gt;CT&lt;/sub&gt;</td>
<td>54.00 kip</td>
</tr>
<tr>
<td>Collision Load Distribution L&lt;sub&gt;L&lt;/sub&gt;</td>
<td>3.50 ft.</td>
</tr>
<tr>
<td>Top of wall to point of collision impact on rail h&lt;sub&gt;CT&lt;/sub&gt;</td>
<td>2.67 ft.</td>
</tr>
</tbody>
</table>

1. STABILITY CHECKS

Use the load combinations and factors from AASHTO 11.5.6 and BDM Section 11.5.1 for all loads acting on the retaining wall. Evaluate the retaining wall for the following:

1. Eccentricity
2. Sliding
3. Bearing

Note: The Geotechnical Engineer is responsible for evaluating global stability with consideration for both footing width and embedment.

APPLIED LOADS

Loads not listed here may be applicable for different design cases.
DC - dead load of structural components and nonstructural attachments
EH - horizontal earth pressure load
EV - vertical pressure from dead load of earth fill
CT - vehicular collision force
LS - live load surcharge
Figure 1 - Typical Section

See Figure 2 for Shear Key Information
Summary of Unfactored Loads and Moments
Resolve moments about Point A (see Figure 1 - Typical Section)

<table>
<thead>
<tr>
<th>Vertical Loads &amp; Moments</th>
<th>Description</th>
<th>V (kip/ft.)</th>
<th>Moment Arm (ft.)</th>
<th>MV (kip-ft.)/ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC1</td>
<td>Stem dead load</td>
<td>3.38</td>
<td>3.50</td>
<td>11.81</td>
</tr>
<tr>
<td>DC2</td>
<td>Stem dead load</td>
<td>0.28</td>
<td>4.33</td>
<td>1.22</td>
</tr>
<tr>
<td>DC3</td>
<td>Footing dead load</td>
<td>1.88</td>
<td>5.00</td>
<td>9.38</td>
</tr>
<tr>
<td>DC4</td>
<td>Barrier dead load</td>
<td>0.49</td>
<td>3.32</td>
<td>1.61</td>
</tr>
<tr>
<td>EV1</td>
<td>Vertical pressure from dead load of fill on heel</td>
<td>10.73</td>
<td>7.25</td>
<td>77.76</td>
</tr>
<tr>
<td>EV2</td>
<td>Vertical pressure from dead load of fill on heel</td>
<td>0.24</td>
<td>4.42</td>
<td>1.08</td>
</tr>
<tr>
<td>EV3</td>
<td>Vertical pressure from dead load of fill on toe</td>
<td>0.72</td>
<td>1.38</td>
<td>0.98</td>
</tr>
<tr>
<td>EVV</td>
<td>Vertical component of horizontal earth pressure</td>
<td>1.68</td>
<td>10.00</td>
<td>16.82</td>
</tr>
<tr>
<td>LSV</td>
<td>Vertical component of live load surcharge</td>
<td>0.98</td>
<td>8.13</td>
<td>7.92</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Horizontal Loads and Moments</th>
<th>Description</th>
<th>H (kip/ft.)</th>
<th>Moment Arm (ft.)</th>
<th>MH (kip-ft.)/ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>EHV</td>
<td>Horizontal component of horizontal earth pressure</td>
<td>4.03</td>
<td>5.42</td>
<td>21.81</td>
</tr>
<tr>
<td>LSVH</td>
<td>Horizontal component of live load surcharge</td>
<td>1.07</td>
<td>8.13</td>
<td>8.73</td>
</tr>
<tr>
<td>CT</td>
<td>Vehicular collision load</td>
<td>2.61</td>
<td>18.92</td>
<td>49.43</td>
</tr>
</tbody>
</table>

\[
EH_V = \sin(\delta) \times EH = \sin(\delta) \times 0.5 \times EFW(a) \times (H + T_F)^2 \\
EH_H = \cos(\delta) \times EH = \cos(\delta) \times 0.5 \times EFW(a) \times (H + T_F)^2 \\
LS_V = \gamma_s \cdot h_{Sur} \times (B - S - T_{Top} - R) \\
LS_H = EFW(a) \cdot h_{Sur} \times (H + T_F)
\]

Note: The collision force (CT) is assumed to be distributed over a length of “Lt” ft. at the point of impact and is also assumed to spread downward to the bottom of the footing at a 45° angle. Conservatively, CT is assumed at the end of the wall where the force distribution occurs in one direction. See Figure 11-20 in Section 11 of this BDM.

Reinforcement between the Bridge Rail Type 7 and the wall interface is assumed to be adequate to transfer the collision load from the rail through the wall to the footing.

\[
CT = P_{CT} / \left( L_t / 2 + (h_{CT} + H + T_F) \right)
\]

Load Combinations
The table that follows summarizes the load combinations used for the stability and bearing checks of the wall. To check sliding and eccentricity, load combinations Strength Ia and Extreme Event IIa apply minimum load factors to the vertical loads and maximum load factors to the horizontal loads. To check bearing, load combinations Strength Ib, Strength IV, and Extreme Event IIb apply maximum load factors for both vertical and horizontal loads.
CT load is considered with Extreme Event II limit state when checking eccentricity, sliding, and bearing.

**Note:** LS<sub>h</sub>, LS<sub>v</sub>, and EH<sub>h</sub> are not included in Extreme Event Ila or I Ib. It is assumed that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

LS<sub>v</sub> is not applied when analyzing sliding and overturning; rather, it is applied only for load combinations that are used to analyze bearing (AASHTO 11.5.6, Figure C11.5.6-3a).

The service limit state is used for the crack control check and settlement.

**Total factored force effect:**

\[ Q = \Sigma \eta_i \gamma_i Q_i \]

where \( Q_i \) = force effects from loads calculated above

**Load Modifiers:**

- **Ductility** \( \eta_D = 1.00 \) (AASHTO 1.3.3-1.3.5)
- **Redundancy** \( \eta_r = 1.00 \)
- **Operational Importance** \( \eta_I = 1.00 \)

**Load Factors:**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( \gamma_D )</th>
<th>( \gamma_E )</th>
<th>( \gamma_{LS,v} )</th>
<th>( \gamma_{LS,h} )</th>
<th>( \gamma_{EH} )</th>
<th>( \gamma_{CT} )</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength Ia</td>
<td>0.90</td>
<td>1.00</td>
<td>-</td>
<td>1.75</td>
<td>1.50</td>
<td>-</td>
<td>Sliding, Eccentricity</td>
</tr>
<tr>
<td>Strength Ib</td>
<td>1.25</td>
<td>1.35</td>
<td>1.75</td>
<td>1.75</td>
<td>1.50</td>
<td>-</td>
<td>Bearing, Strength Design</td>
</tr>
<tr>
<td>Strength IV</td>
<td>1.50</td>
<td>1.35</td>
<td>-</td>
<td>-</td>
<td>1.50</td>
<td>-</td>
<td>Bearing</td>
</tr>
<tr>
<td>Extreme Ila</td>
<td>0.90</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>Sliding, Eccentricity</td>
</tr>
<tr>
<td>Extreme I Ib</td>
<td>1.25</td>
<td>1.35</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>Bearing</td>
</tr>
<tr>
<td>Service I</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>-</td>
<td>Wall Crack Control</td>
</tr>
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</table>

**Summary of Load Groups:**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Vertical Load &amp; Moment</th>
<th>Horizontal Load &amp; Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( V ) (kip/ft.)</td>
<td>( MV ) (kip-ft./ft.)</td>
</tr>
<tr>
<td>Strength Ia</td>
<td>19.62</td>
<td>126.66</td>
</tr>
<tr>
<td>Strength Ib</td>
<td>27.52</td>
<td>176.87</td>
</tr>
<tr>
<td>Strength IV</td>
<td>27.32</td>
<td>169.01</td>
</tr>
<tr>
<td>Extreme Ila</td>
<td>17.10</td>
<td>101.43</td>
</tr>
<tr>
<td>Extreme I Ib</td>
<td>23.29</td>
<td>137.78</td>
</tr>
<tr>
<td>Service I</td>
<td>20.36</td>
<td>128.58</td>
</tr>
</tbody>
</table>
Eccentricity (Overturning) Check

When a shear key is required to prevent sliding, the passive resistance shall be ignored.

Maximum eccentricity limit: \( e_{\text{max}} = \frac{B}{3} = 3.33 \text{ ft.} \) AASHTO 10.6.3.3

\[
e_{\text{actual}} = \frac{B}{2} - \frac{\Sigma M_V - \Sigma M_H}{\Sigma V}
\]

Strength Ia: \[
X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} = \frac{(126.66 - 47.99)}{19.62} = 4.01 \text{ ft.}
\]
\[
e = \frac{10.0}{2} - 4.01 = 0.99 \text{ ft.} \quad e_{\text{actual}} < e_{\text{max}} \quad \text{OK}
\]

Extreme Ila: \[
X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} = \frac{(101.43 - 49.43)}{17.10} = 3.04 \text{ ft.}
\]
\[
e = \frac{10.0}{2} - 3.04 = 1.96 \text{ ft.} \quad e_{\text{actual}} < e_{\text{max}} \quad \text{OK}
\]

Bearing Resistance Check

When a shear key is required to prevent sliding, the passive resistance shall be ignored.

Vertical stress for wall supported on soil: \( \sigma_v = \frac{\Sigma V}{B - 2e} \) AASHTO 11.6.3.2-1

Nominal soil bearing resistance \( q_n = 7.50 \text{ ksf} \)
Factored bearing resistance \( q_R = \phi_b q_n = 4.13 \text{ ksf} \)
\( q_{R,\text{EE}} = \phi_{\text{EE}} q_n = 7.50 \text{ ksf} \) Extreme event

Strength Ib: \[
X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} = \frac{(176.87 - 47.99)}{27.52} = 4.68 \text{ ft.}
\]
\[
e = \frac{B}{2} - X = \frac{10.0}{2} - 4.68 = 0.32 \text{ ft.} \quad \sigma_v < q_R \quad \text{OK}
\]

Strength IV: \[
X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} = \frac{(169.01 - 32.72)}{27.32} = 4.99 \text{ ft.}
\]
\[
e = \frac{B}{2} - X = \frac{10.0}{2} - 4.99 = 0.01 \text{ ft.} \quad \sigma_v < q_R \quad \text{OK}
\]

Extreme Iib: \[
X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} = \frac{(137.78 - 49.43)}{23.29} = 3.79 \text{ ft.}
\]
\[
e = \frac{B}{2} - X = \frac{10.0}{2} - 3.79 = 1.21 \text{ ft.} \quad \sigma_v < q_{R,\text{EE}} \quad \text{OK}
\]
Sliding Check

Per AASHTO 10.6.3.5, passive soil pressure shall be neglected.

Strength la and Extreme Ila:

- Maximum total Horizontal force \( \Sigma H = 7.92 \text{ kip/ft.} \)
- Maximum total Vertical force \( \Sigma V = 19.62 \text{ kip/ft.} \)
- Nominal passive resistance \( R_{ep} = 0.00 \text{ kip/ft.} \)
- For concrete cast against soil \( C = 1.00 \text{ kip/ft.} \)
- Nominal soil sliding coefficient \( \mu_n = \tan \phi_{sub} = 0.364 \)
- Nominal sliding resistance \( R_f = C \Sigma V \mu_n = 1.0 (19.62) (0.364) = 7.14 \text{ kip/ft.} \)
- Factored resistance against failure by sliding
  \[ R_R = \phi R_n = \phi_T R_T + \phi_{ep} R_{ep} = 1.00 (7.14) + 0.50 (0.0) = 7.14 \text{ kip/ft.} \]
  \[ R_R < \Sigma H \quad \text{Shear Key is Required} \]

Shear Key Design

1. Assume shear key dimensions.
2. Center line of the shear key is approximately B/3 from the heel edge of the footing; see BDM Section 1
3. Passive soil pressure at the toe shall be neglected; only include passive pressure due to the inert block (c) (see AASHTO 11.6.3.5).
4. Depth of inert block is taken to be the sum of the key depth and the effective wedge depth. This example follows this methodology. Conservatively, effective wedge depth can be ignored, allowing inert block to be equal to shear key depth.
5. Per BDM Section 11.5.1, the top 1 ft. of fill at the toe shall be ignored for all design cases.
6. The Designer may choose to add weight of the shear key for eccentricity and bearing analysis once shear key dimensions are confirmed. For this example, weight of the key is ignored.

Shear key depth \( d_{Key} = 1.00 \text{ ft.} \)
Shear key width \( T_{Key} = 1.50 \text{ ft.} \)
Heel of footing to centerline shear key \( K = 3.50 \text{ ft.} \)
Toe of footing to front face of shear key \( X_{Key} = 5.75 \text{ ft.} \)
Soil cover above the footing toe \( H_{TF} = 2.00 \text{ ft.} \)
Shear friction angle of subgrade \( \delta_{sub} = 2/3\phi_{sub} = 13.33 \text{ deg.} \)
Inert block depth \( c = d_{Key} + X_{Key} \tan(\delta_{sub}) = 2.36 \text{ ft.} \)
Top of fill to top of shear key \( y_1 = 2.25 \text{ ft.} \)
Top of fill to bottom of inert block \( y_2 = 4.61 \text{ ft.} \)

Passive equivalent pressure \( EFW(p) = 0.988 \text{ kcf} \)
Nominal soil sliding coefficient \( \mu_n = 0.364 \)

Coefficients of friction (factored):
- Concrete-soil \( \mu_{c} = \phi_T \mu_n = 1.00 (0.364) = 0.364 \)
- Soil-soil \( \mu_{ss} = \phi_{Ts} \mu_n = 1.00 (0.364) = 0.364 \)
- Extreme event \( \mu_{EE} = \phi_{EE} \mu_n = 1.00 (0.364) = 0.364 \)
EXAMPLE 11 - CAST-IN-PLACE CONCRETE CANTILEVER RETAINING WALL

Shear resistance between soil and foundation:

- Strength Ia:
  \[ \phi R_t = C \, R_1 \, \mu_{u,s-s} \cos \delta_{sub} + C \, R_2 \, \mu_u \]  
  (Strength Ia)

- Extreme IIa:
  \[ \phi_{EE} R_t = C \, R_1 \, \mu_{u,EE} \cos \delta_{sub} + C \, R_2 \, \mu_u \, EE \]  
  (Extreme IIa)

\[ X = \frac{(\Sigma M_V - \Sigma M_H)}{\Sigma V} \]
\[ e = \frac{B}{2} - X \]
\[ \sigma_v = \frac{\Sigma V}{B - 2e} \]

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( \Sigma V ) (kip/ft.)</th>
<th>( \Sigma M_V ) (kip-ft./ft.)</th>
<th>( \Sigma M_H ) (kip-ft./ft.)</th>
<th>( X ) (ft.)</th>
<th>( e ) (ft.)</th>
<th>( \sigma_v ) (ksf)</th>
<th>( R_1 ) (kip/ft.)</th>
<th>( R_2 ) (kip/ft.)</th>
<th>( \phi R_t ) (kip/ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength Ia</td>
<td>19.62</td>
<td>126.66</td>
<td>47.99</td>
<td>4.01</td>
<td>0.99</td>
<td>2.45</td>
<td>14.07</td>
<td>10.40</td>
<td>8.77</td>
</tr>
<tr>
<td>Extreme IIa</td>
<td>17.10</td>
<td>101.43</td>
<td>49.43</td>
<td>3.04</td>
<td>1.96</td>
<td>2.81</td>
<td>16.16</td>
<td>11.95</td>
<td>10.07</td>
</tr>
</tbody>
</table>

Passive resistance of soil available throughout the design life of structure:
\[ R_{ep} = EFW \,(p)0.5 \,(y_1 + y_2) \, c = 0.988 \times 0.5 \,(2.25 + 4.61) \, 2.36 = 8.01 \, \text{kip} \]

Factored resistance against failure by sliding:

- Strength Ia:
  Maximum total Horizontal force \( \Sigma H = 7.92 \, \text{kip} \)
  \[ R_R = \phi R_n = \phi_t R_t + \phi_{ep} R_{ep} = 8.77 + 0.50 \,(8.01) = 12.77 \, \text{kip} \]
  \[ R_R > \Sigma H \quad \text{OK} \]

- Extreme IIa:
  Maximum total Horizontal force \( \Sigma H = 2.61 \, \text{kip} \)
  \[ R_R = \phi R_n = \phi_{EE} R_t + \phi_{ep} R_{ep} = 10.07 + 0.50 \,(8.01) = 14.08 \, \text{kip} \]
  \[ R_R > \Sigma H \quad \text{OK} \]

Figure 2 - Shear Key
2. STRENGTH DESIGN

Concrete compressive strength \( f'_c = 4.50 \) ksi
Yield strength of the reinforcement \( f_y = 60.00 \) ksi
Concrete unit weight \( \gamma_c = 0.150 \) kcf
Correction factor for source aggregate \( K_1 = 1.00 \) AASHTO 5.4.2.4
Modulus of elasticity of reinforcement \( E_s = 29000 \) ksi AASHTO 5.4.3.2
Modulus of elasticity of concrete \( E_c = 120,000K_1\gamma_c^2f'_c^{0.33} = 4435.31 \) ksi AASHTO 5.4.2.4
Modular ratio \( n = \frac{E_s}{E_c} = 6.54 \) AASHTO 5.7.1
Compression zone factor \( \beta_1 = 0.85 - (f'_c - 4.0)0.05 = 0.825 \) AASHTO 5.7.2.2
Resistance factor for flexural-tension control \( \phi_f = 0.90 \) AASHTO 5.5.4.2.1
Resistance factor for shear-tension control \( \phi_v = 0.90 \) AASHTO 5.5.4.2.1
Design width \( b = 12.00 \) in.

2.1 STEM WALL DESIGN

Summary of unfactored horizontal loads and moments at the bottom of the stem:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
<th>( H ) (kip/ft.)</th>
<th>Moment Arm (ft.)</th>
<th>MH (kip-ft./ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EH(_H)</td>
<td>Soil</td>
<td>3.43</td>
<td>5.00</td>
<td>17.16</td>
</tr>
<tr>
<td>LS(_H)</td>
<td>Surcharge</td>
<td>0.99</td>
<td>7.50</td>
<td>7.44</td>
</tr>
</tbody>
</table>

Summary of Load Groups:

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Horizontal Load &amp; Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vu (kip/ft.)</td>
</tr>
<tr>
<td>Strength I lb</td>
<td>6.88</td>
</tr>
<tr>
<td>Service I</td>
<td>4.42</td>
</tr>
</tbody>
</table>

It has been assumed that the load combination Strength I lb generates the maximum moment at the interface of the stem wall and footing. However, the Designer should check all possible load combinations, including extreme event, and select the combination that produces the maximum load for the design of the stem.

**Note:** The Designer/Engineer is encouraged to use engineering judgment to determine the moment and required area of reinforcing steel at other points of the stem for tall walls (\( H \geq 10.0'\)) to reduce the amount of steel required at higher elevations.
2.1.1 Flexure Design

Design of vertical reinforcement bars at back face of stem

Assumed bar size  \( \text{Bar} = \# 5 \)
Factored applied moment  \( M_{u, \text{Str}} = 38.75 \text{ kip-ft. / ft.} \)
Concrete clear cover  \( r = 2.00 \text{ in.} \)
Bar diameter  \( d_b = 0.625 \text{ in.} \)
Bar area  \( A_b = 0.310 \text{ in}^2 \)
Effective Depth  \( d_e = T_{\text{Bot}} - r - d_b / 2 = 1.75' (12) - 2'' - 0.625'' / 2 = 18.69 \text{ in.} \)

Try  \( \# 5 \) @ 6.0” on center:

Design steel area  \[ A_s = A_b \frac{b}{\text{spa}} = 0.310 (12) / 6 = 0.620 \text{ in}^2 \]
Distance from compression fiber to neutral axis  \[ c_b = \frac{A_s f_y}{\beta f_{y,c} b} = 0.620 (60) / (0.825*0.85*4.5*12) = 0.982 \text{ in.} \]
Equivalent Stress Block  \[ a = \beta \frac{f_y}{c_b} = 0.825 (0.982) = 0.810 \text{ in.} \]
Nominal Flexural Resistance  \[ M_n = A_s f_y \left( d_e - \frac{a}{2} \right) = 0.620 (60) (18.69 - 0.810 / 2) = 56.68 \text{ kip-ft.} \]
Factored Flexural Resistance  \[ M_R = \phi_f M_n = 0.90 (56.68) = 51.01 \text{ kip-ft.} \]
\[ M_R > M_{u, \text{Str}} \text{ OK} \]

Maximum Reinforcement:  Provision deleted in 2005  AASHTO 5.7.3.3.1

Minimum Reinforcement:  AASHTO 5.7.3.3.2
The amount of tensile reinforcement shall be adequate to develop a factored flexural resistance, \( M_{kr} \), at least equal to the lesser of 1.33\( M_{u, \text{Str}} \) or \( M_{cr} \)

Member width  \( b = 12.00 \text{ in.} \)
Member depth  \( d = T_{\text{Bot}} = 21.00 \text{ in.} \)
Distance to Neutral Axis  \( y_t = T_{\text{Bot}} / 2 = 10.50 \text{ in.} \)
Stem moment of inertia  \[ I_g = b d^3 / 12 = 9261.0 \text{ in}^4 \]
Section modulus  \[ S_{nc} = S_c = I_g / y_t = 882.0 \text{ in}^3 \]
Concrete Modulus of Rupture  \[ f_r = 0.20 \sqrt{f'_c} = 0.424 \text{ ksi} \]  AASHTO 5.4.2.6
EXAMPLE 11 - CAST-IN-PLACE CONCRETE CANTILEVER RETAINING WALL

11

Cracking moment, 

\[ M_{cr} = y_3 \left[ (y_1 f_r + y_2 f_{c pe}) S_c - M_{dn c} (y_c/y_{nc} - 1) \right] \]  

AASHTO 5.7.3.3.2-1

Flexural cracking variability factor

\[ y_1 = 1.600 \]

Prestress variable factor

\[ y_2 = 0.000 \]

Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement

\[ y_3 = 0.670 \] for A615, Grade 60 steel

Compressive stress due to prestress force

\[ f_{c pe} = 0.000 \text{ ksi} \]

Total unfactored dead load moment

\[ M_{dn c} = 0.000 \text{ kip-in.} \]

Cracking moment,

\[ M_{cr} = 0.670 \left[ (1.60 * 0.424 + 0) * 882.0 - 0 \right] / 12 = 33.43 \text{ kip-ft./ft. - controls} \]

Factored applied moment *1.33

\[ 1.33 M_{u Str} = 51.54 \text{ kip-ft./ft.} \]

Factored flexural resistance

\[ M_R = 51.01 \text{ kip-ft./ft.} \]

\[ M_R > \min (M_{cr}, 1.33M_{u Str}) \quad \text{OK} \]

Control of cracking by distribution of reinforcement:

AASHTO 5.7.3.4

Exposure condition class

Use Class 2 for the stem, Class 1 for the footing and key

Exposure factor

\[ \gamma_e = 0.75 \]

Thickness of concrete cover

\[ d_c = 2^* d_b / 2 = 2^* + 0.625 / 2 = 2.31 \text{ in.} \]

Reinforcement Ratio

\[ \rho = A_s / b d_e = 0.620 / (12 * 18.69) = 0.003 \]

Modular ratio

\[ n = 6.54 \]

\[ k = \sqrt{2n\rho + (n\rho)^2} - n\rho = 0.173 \]

\[ j = 1 - k / 3 = 0.942 \]

Service applied moment

\[ M_{u serv} = 24.59 \text{ kip-ft.} \]

Tensile stress in steel

\[ f_{ss} = M_{u serv} * 12 / (A_s / d_e) = 24.59(12) / (0.620*0.942*18.69) = 27.03 \text{ ksi} \]

\[ \beta_s = 1 + \frac{d_c}{0.7(T_{Bot} - d_c)} = 1 + 2.31 / 0.7 (1.75*12 - 2.31) = 1.18 \]

Maximum spacing

\[ s_{max} = \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c = 700 (0.75) / (1.18*27.03) - 2(2.31) = 11.88 \text{ in.} \]

Spacing provided

\[ s_{prov} = 6.00 \text{ in.} \]

\[ s_{prov} < s_{max} \quad \text{OK} \]

2.1.2 Shear Design

AASHTO 5.8.3.3

Shear typically does not govern the design of retaining walls. If shear becomes an issue, the thickness of the stem should be increased. Ignore benefits of the shear key (if applicable) and axial compression.

Factored shear load

\[ V_{u str} = 6.88 \text{ kip/ft.} \]

Shear resistance Factor

\[ \beta = 2.00 \] AASHTO 5.8.3.4

Concrete density modification factor

\[ \lambda = 1.00 \] AASHTO 5.4.2.8

Effective Depth

\[ d_v = \max (d_a - C_v/2, 0.9 d_e, 0.72 T_{Bot}) = 18.20 \text{ in (shear)} \] AASHTO 5.8.2.9

Nominal Shear Resistance

\[ V_c = 0.0316\beta\lambda\sqrt{f'_c} b d_v = 0.0316 (2)(1)\sqrt{4.50} (12)(18.20) = 29.27 \text{ kip} \]

Factored Shear Resistance

\[ V_R = \phi V_c = 0.90 (29.27) = 26.35 \text{ kip} \]
Retaining wall footings and stems are typically unreinforced for shear. Confirm transverse reinforcement is not required by design, \( 0.5 V_R > V_{u, str} \) AASHTO 5.8.2.4

\[
0.5 V_R = 13.17 \text{ kip}
\]

\[0.5 V_R > V_{u, str} \quad \text{OK}\]

### 2.1.3 Shrinkage and Temperature Reinforcement Design AASHTO 5.10.8

Horizontal reinforcement at each face of stem and vertical reinforcement at front face of stem

**Try # 4 @ 12.0” on center:**

Design steel area \( A_S = 0.200 \text{ in}^2 \)

Check \( A_S \geq \frac{1.30 b T_{Bot}}{2(b + T_{Bot})f_y} = 0.083 \text{ in}^2 \quad \text{OK} \)

Check \( 0.11 \leq A_s \leq 0.60 \quad \text{OK} \)

### 2.2 FOOTING HEEL DESIGN

The critical section for shear and moment is at the back face of the stem wall (C5.13.3.6). The heel is designed to carry its self weight and the soil block above it. Conservatively, it is common to ignore upward soil reaction under the footing heel, thus Strength 1b is not checked. For shear in footings, the provisions of 5.8.2.4 are not applicable, thus \( \phi V_c \geq V_u \).

Summary of Unfactored Vertical Loads and Moments at the Back Face of the Stem:

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Description</th>
<th>( V ) (kip/ft.)</th>
<th>Moment Arm (ft.)</th>
<th>( M ) (kip-ft.)/ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC</td>
<td>Heel dead load</td>
<td>1.03</td>
<td>2.75</td>
<td>2.84</td>
</tr>
<tr>
<td>EV_1</td>
<td>Vertical pressure from dead load of fill on heel</td>
<td>10.73</td>
<td>2.75</td>
<td>29.49</td>
</tr>
</tbody>
</table>

Summary of Load Groups:

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Vertical Load &amp; Moment</th>
<th>( V_u ) (kip/ft.)</th>
<th>( M_u ) (kip-ft.)/ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength IV</td>
<td>16.03</td>
<td>44.07</td>
<td></td>
</tr>
<tr>
<td>Service I</td>
<td>11.76</td>
<td>32.33</td>
<td></td>
</tr>
</tbody>
</table>

By inspection, load combination Strength IV generates a maximum moment at the interface of the footing heel and stem wall. However, the Designer should check all possible load combinations and select the combination that produces the maximum load for the design of the footing.

For reinforcement design, follow the procedure outlined in Section 2.1. Exposure Class I can be used for cracking check. Results of the design are as follows (also shown on Figure 3):

Transverse horizontal bar at top of footing - \(# 6 @ 6.0”\)

Longitudinal reinforcement, top and bottom of footing - \(# 4 @ 12.0”\)
2.3 FOOTING TOE DESIGN

The critical section for shear is \( d_y \) from front face of wall stem and, for moment, is at the front face of wall stem (C5.13.3.6). Section is designed to resist bearing stress acting on toe. This example conservatively ignores the soil on top of the toe. For shear in footings, the provisions of 5.8.2.4 are not applicable, thus \( \phi V_c \geq V_u \).

Controlling loads:

| Maximum bearing stress (factored) | \( \sigma_y \) = 3.07 ksf (from bearing resistance check) |
| Factored shear | \( V_{u,\text{str}} = \sigma_y S = 8.45 \text{ kip/ft.} \) |
| Factored bending moment | \( M_{u,\text{str}} = V_u S/2 = 11.61 \text{ kip/ft.} \) |

Service loads:

\[
X = \frac{\sum M_V - \sum M_h}{\sum V} = \frac{(128.58 - 30.54)}{20.36} = 4.82 \text{ ft.}
\]

\[
e = \frac{B}{2} - X = \frac{10.0}{2} - 4.82 = 0.18 \text{ ft.}
\]

\[
\sigma_y = \frac{\Sigma V}{(B - 2e)} = \frac{20.36}{(10.0 - 2(0.18))} = 2.11 \text{ ksf}
\]

| Factored shear | \( V_{u,\text{serv}} = \sigma_y S = 5.81 \text{ kip/ft.} \) |
| Factored bending moment | \( M_{u,\text{serv}} = V_u S/2 = 7.99 \text{ kip/ft.} \) |

For reinforcement design, follow the procedure outlined in Section 2.1. Results of the design are as follows (also shown on Figure 3):

Transverse horizontal bar at bottom of toe - \# 5 @ 6.0"

Note: Check that the toe length and footing depth can accommodate development length of the hooked bar past the design plane.

2.4 SHEAR KEY DESIGN

The critical section for shear and moment is at the interface with the bottom of the footing. Shear key reinforcing is designed to resist passive pressure determined in the sliding analysis. Passive pressure load resultant is located at a distance "z" from the bottom of footing, if using inclined wedge (see Figure 2).

\[
R_{ep} = 8.01 \text{ kip}
\]

\[
R_{ep} = \frac{[0.5 K_p y_c c^2 + 0.333 K_p y_c^3]}{8.01} = \frac{[0.5 (7.60)(0.130)(2.25)(2.36)^2 + 0.333 (7.60)(0.130)(2.36)^3]}{8.01} = 1.32 \text{ ft.}
\]

Factored bending moment for key design \( M_{u,\text{str}} = 10.55 \text{ kip-ft.} \)

For reinforcement design, follow the procedure outlined in Section 2.1. Results of the design are as follows (also shown on Figure 3):

Vertical "U" bars at front and back face of shear key - \# 4 @ 6.0"

Longitudinal reinforcement in shear key - \# 4 @ 12.0"
Figure 3 - Final Wall Section