Design of a Balanced-Cantilever Bridge

CL (Bridge is symmetric about CL)

L = 80 ft ⇒ Bridge Span = 2.6 L = 2.6 × 80' = 208'
Bridge Width = 30'
No. of girders = 6, Width of each girder = 15"
Loads: LL = HS20, Wearing Surface = 30 psf
Material Properties: \( f_c = 3 \, \text{ksi}, f_s = 20 \, \text{ksi} \)
1. Design of Slab

Bridge width = 30', Number of girders = 6@15''

\[ \text{Clear span between girders } S = (30' - 6 \times 15'')/5 = 4.5' \]

\[ \text{The c/c distance between girders } = 4.5' + 15'' = 5.75' \]

Assuming slab thickness = 6'' \Rightarrow \text{Slab weight} = 75 \text{ psf} = 0.075 \text{ ksf}

\[ \text{Wearing Surface} = 30 \text{ psf} \Rightarrow w_{DL} = 75 + 30 = 105 \text{ psf} = 0.105 \text{ ksf} \]

\[ \text{Dead-load moment, } M_{DL} = w_{DL} S^2/10 = 0.105 \times 4.5^2/10 = 0.213 \text{ k}'/'' \]

AASHTO specifies Live-load moment, \[ M_{LL} = 0.8 (S+2)/32 P_{20} \]

\[ = 0.8 \times [(4.5+2)/32] \times 16 = 2.6 \text{ k}'/'' \]

Impact factor, \[ I = 50/(S+125) = 0.386 > 0.3 \Rightarrow I = 0.3 \]

Impact moment, \[ M_{IMP} = M_{LL} \times I = 0.213 \times 0.3 = 0.78 \text{ k}'/'' \]

\[ \text{Total moment, } M_T = M_{DL} + M_{LL} + M_{IMP} = 0.213 + 2.6 + 0.78 = 3.593 \text{ k}'/'' \]

For design, \[ f_c' = 3 \text{ ksi} \Rightarrow f_c = 0.4 f_c' = 1.2 \text{ ksi} \]

\[ n = 9, k = 9/(9+20/1.2) = 0.351, j = 1 - k/3 = 0.883 \]

\[ R = \frac{1}{2} \times 1.2 \times 0.351 \times 0.883 = 0.186 \text{ ksi} \]

\[ d_{req} = \sqrt{(M_T/Rb)} = \sqrt{(3.593/0.186 \times 1)} = 4.40'' \]

\[ t = 6'' \Rightarrow d = 6'' - 1.5'' = 4.5'' > d_{req} \]

\[ d = 4.5'', t = 6'' \text{ (OK)}. \]

\[ \text{Required reinforcement, } A_s = M_T/(f_c j d) = 3.593 \times 12/(20 \times 0.883 \times 4.5) = 0.543 \text{ in}^2/'' \]

\[ \text{Use #5 @7'' c/c [or #6 @10'' c/c]} \]

Also, \[ 2.2/\sqrt{S} = 2.2/\sqrt{4.5} = 1.04 > 0.67 \]

\[ \text{Distribution steel, } A_{s(dist)} = 0.67 A_s = 0.67 \times 0.543 = 0.364 \text{ in}^2/'' \]

\[ A_{s(dist)} \text{ per c/c span} = 5.75' \times 0.364 \text{ in}^2/'' \]

\[ = 2.09 \text{ in}^2; \text{ i.e., 7 #5 bars (to be placed within the clear spans).} \]
2. Dead Load Analysis of Interior Girders

Girder depths remain constant between A-D and vary parabolically between D-I and I-N.

The variation is symmetric about I. If the girder depths at D and N are both 40" (L/2 in inches) and that at I is 70" (about 70-80% larger), the depths at the other sections can be calculated easily. The depths calculated are the following:

\[ h_A = 40", \ h_B = 40", \ h_C = 40", \ h_D = 40", \ h_E = 41.2", \ h_F = 44.8", \ h_G = 50.8", \ h_H = 59.2", \ h_I = 70", \ h_J = 59.2", \ h_K = 50.8", \ h_L = 44.8", \ h_M = 41.2", \ h_N = 40" \]

Using these dimensions (with an additional 30 psf; i.e., 2.4" concrete layer) for the analysis of the girder for self-weight using the software GRASP, the following results are obtained.

**Table 2.1 Dead Load Shear Forces and Bending Moments**

<table>
<thead>
<tr>
<th>Section</th>
<th>x from left (')</th>
<th>h (&quot;)</th>
<th>V (k)</th>
<th>M (k')</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0</td>
<td>40</td>
<td>27.40</td>
<td>0.00</td>
</tr>
<tr>
<td>B</td>
<td>8</td>
<td>40</td>
<td>18.32</td>
<td>182.84</td>
</tr>
<tr>
<td>C</td>
<td>16</td>
<td>40</td>
<td>9.24</td>
<td>293.04</td>
</tr>
<tr>
<td>D</td>
<td>24</td>
<td>40</td>
<td>0.16</td>
<td>330.59</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>41.2</td>
<td>-9.00</td>
<td>295.21</td>
</tr>
<tr>
<td>F</td>
<td>40</td>
<td>44.8</td>
<td>-18.46</td>
<td>185.39</td>
</tr>
<tr>
<td>G</td>
<td>48</td>
<td>50.8</td>
<td>-28.51</td>
<td>-2.48</td>
</tr>
<tr>
<td>H</td>
<td>56</td>
<td>59.2</td>
<td>-39.47</td>
<td>-274.39</td>
</tr>
<tr>
<td>I (L)</td>
<td>64</td>
<td>70</td>
<td>-51.62</td>
<td>-638.72</td>
</tr>
<tr>
<td>I (R)</td>
<td>64</td>
<td>70</td>
<td>51.78</td>
<td>-638.72</td>
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<tr>
<td>J</td>
<td>72</td>
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<td>39.62</td>
<td>-273.14</td>
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<td>50.8</td>
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<td>L</td>
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<td>44.8</td>
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<td>189.10</td>
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<td>M</td>
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<td>41.2</td>
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<td>300.16</td>
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<tr>
<td>N</td>
<td>104</td>
<td>40</td>
<td>0.00</td>
<td>336.78</td>
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</tbody>
</table>
3. Live Load Analysis of Interior Girders

The live load analysis of interior girders is carried out for HS20 loading with wheel loads of 4 k, 16 k, and 16 k at 14′ distances, as shown below.

For live-load analysis, each wheel load (4 k, 16 k, 16 k) needs to be multiplied by a factor

\[ S/5 \geq 1.0 \]

In this case, \( S = 5.75 \); \( \therefore \) Factor = \( 5.75/5 = 1.15 > 1.0 \), OK

\( \therefore \) Wheel Loads for live load analysis are

\[ (16 \times 1.15) = 18.4 \text{ k}, \quad (16 \times 1.15) = 18.4 \text{ k} \quad \text{and} \quad (4 \times 1.15) = 4.6 \text{ k} \]

Also, the impact factor \( I = \frac{50}{(L_0+125)} \leq 0.30 \), where \( L_0 = \) Loaded length

Assuming \( L_0 = 0.6L = 48′ \) (conservatively), \( I = \frac{50}{(48+125)} = 0.289 \)

\( \therefore \) The impact shear forces and bending moments can be obtained by multiplying live load shears and moments by \( I (= 0.289) \).

As an alternative to using separate moments for live load and impact, one can do them simultaneously by multiplying the wheel loads by \( (1 + I) = 1.289 \); i.e., taking wheel loads to be

\[ (18.4 \times 1.289) = 23.72 \text{ k}, \quad (18.4 \times 1.289) = 23.72 \text{ k} \quad \text{and} \quad (4.6 \times 1.289) = 5.93 \text{ k} \]

The combined (live load + impact) shears and bending moments can be obtained by moving the wheels from A to N (keeping \( W_1 \) or \( W_3 \) in front) and recording all the shear forces (V) and bending moments (M). The software GRASP can be used for this purpose.

Instead of such random wheel movements, Influence Lines can be used to predict the critical position of wheels in order to get the maximum forces. This can considerably reduce the computational effort. The subsequent discussions follow this procedure.
The IL for V and M at the ‘simply supported span’ K-L-M-N and the critical wheel arrangements are as follows.

\[ V_{LL+IMP}(x_s) = \frac{23.72}{L_s} \left[ (L_s - x_s) + (L_s - x_s - 14) + \frac{(L_s - x_s - 28)}{4} \right] \]

\[ M_{LL+IMP}(x_s) = x_s V_{LL+IMP}(x_s); \text{ if } x_s \leq \frac{L_s}{3}. \]

Using these equations, with \( L_s = 48' \), the following values are obtained

[These calculations can be carried out conveniently in EXCEL]

<table>
<thead>
<tr>
<th>Section</th>
<th>( x_s (') )</th>
<th>( V ) (k)</th>
<th>( M ) (k')</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>0</td>
<td>42.99</td>
<td>0.00</td>
</tr>
<tr>
<td>L</td>
<td>8</td>
<td>34.10</td>
<td>272.78</td>
</tr>
<tr>
<td>M</td>
<td>16</td>
<td>25.20</td>
<td>403.24</td>
</tr>
<tr>
<td>N</td>
<td>24</td>
<td>16.81</td>
<td>432.89</td>
</tr>
</tbody>
</table>
The IL for \( V \) and \( M \) at the span ‘cantilever span’ I(R)-J-K and the critical wheel arrangements are as follows.

\[
\begin{align*}
23.72 \text{ k} & \quad 23.72 \text{ k} \\
1 & \quad 5.93 \text{ k}
\end{align*}
\]

Using \(< x > = x\), if \( x \geq 0 \), or \( = 0 \) otherwise

\[
\begin{align*}
V_{\text{LL+IMP}}(x_c) &= 23.72 \left[ 1 + \left\{ 1 - < 14 - x_c > / L_\alpha \right\} + \left\{ 1 - (28 - x_c) / L_\alpha \right\} / 4 \right] \\
&= 23.72 \left[ 2.25 - \left\{ < 14 - x_c > + (28 - x_c) / 4 \right\} / L_\alpha \right]
\end{align*}
\]

\[
\begin{align*}
M_{\text{LL+IMP}}(x_c) &= -23.72 \left( x_c / L_\alpha \right) \left[ L_\alpha + (L_\alpha - 14) + (L_\alpha - 28) / 4 \right] \\
&= -(23.72 \times c) \left[ 2.25 - 21 / L_\alpha \right]
\end{align*}
\]

Using these equations, with \( L_\alpha = 48' \), the following values are obtained

**Table 3.2** \( V_{\text{LL+IMP}} \) and \( M_{\text{LL+IMP}} \) for I(R)-J

<table>
<thead>
<tr>
<th>Section</th>
<th>( x_c ) (')</th>
<th>( V ) (k)</th>
<th>( M ) (k')</th>
</tr>
</thead>
<tbody>
<tr>
<td>I (R)</td>
<td>16</td>
<td>51.89</td>
<td>-687.88</td>
</tr>
<tr>
<td>J</td>
<td>8</td>
<td>47.93</td>
<td>-343.94</td>
</tr>
</tbody>
</table>
The IL for V and M at the ‘end span’ A-B-C-D-E-G-H-I(L) and the critical wheel arrangements are as follows.

\[ V_{\text{LL+IMP}}(x_e) = (23.72/L_e) [(L_e-x_e) + (L_e-x_e-14) + (L_e-x_e-28)/4] \geq 0 \]
\[ M_{\text{LL+IMP}}(x_e) = x_e V_{\text{LL+IMP}}(x_e); \text{if } x_e \leq L_e/3. \]
\[ = (23.72/L_e) [x_e (L_e-x_e) + x_e (L_e-x_e-14) + (x_e-14) (L_e-x_e)/4]; \text{otherwise.} \]

For negative shear and moment
\[ V_{\text{LL+IMP}}(x_e) = - (23.72/L_e) [x_e + (x_e-14) + (x_e-28)/4] \leq 0 \]
\[ \text{or} \quad = - (23.72 L_e/L_c) [1 + (1-14/L_a) + (1-28/L_a)/4] \]
\[ M_{\text{LL+IMP}}(x_e) = -(23.72 x_e L_e/L_c) [1 + (1-14/L_a) + (1-28/L_a)/4] \]
Using the derived equations, with $L_c = 16'$ & $L_e = 64'$, the following values are obtained

**Table 3.3 $V_{LL+IMP}$ and $M_{LL+IMP}$ for A-I(L)**

<table>
<thead>
<tr>
<th>Section</th>
<th>$x_0$ ($)</th>
<th>$V^+(k)$</th>
<th>$M^+(k')$</th>
<th>$V^-(k)$</th>
<th>$M^-(k')$</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>0</td>
<td>45.59</td>
<td>0.00</td>
<td>-10.75</td>
<td>0.00</td>
</tr>
<tr>
<td>B</td>
<td>8</td>
<td>38.92</td>
<td>311.33</td>
<td>-10.75</td>
<td>-85.99</td>
</tr>
<tr>
<td>C</td>
<td>16</td>
<td>32.24</td>
<td>515.91</td>
<td>-10.75</td>
<td>-171.97</td>
</tr>
<tr>
<td>D</td>
<td>24</td>
<td>25.57</td>
<td>624.13</td>
<td>-12.23</td>
<td>-257.96</td>
</tr>
<tr>
<td>E</td>
<td>32</td>
<td>18.90</td>
<td>646.37</td>
<td>-18.90</td>
<td>-343.94</td>
</tr>
<tr>
<td>F</td>
<td>40</td>
<td>*</td>
<td>624.13</td>
<td>-25.57</td>
<td>-429.93</td>
</tr>
<tr>
<td>G</td>
<td>48</td>
<td>*</td>
<td>515.91</td>
<td>-32.24</td>
<td>-515.91</td>
</tr>
<tr>
<td>H</td>
<td>56</td>
<td>*</td>
<td>311.33</td>
<td>-38.92</td>
<td>-601.90</td>
</tr>
<tr>
<td>I(L)</td>
<td>64</td>
<td>*</td>
<td>0.00</td>
<td>-45.59</td>
<td>-687.88</td>
</tr>
</tbody>
</table>
The ‘simply supported span’ K-L-M-N

V_{LL+IMP}(x_c) = (23.72/L_x) [(L_c-x_c) + (L_c-x_c-14) + (L_c-x_c-28)/4]

M_{LL+IMP}(x_c) = x_c V_{LL+IMP}(x_c); if x_c ≤ L_x/3,

= (23.72/L_x) [x_c (L_c-x_c) + x_c (L_c-x_c-14) + (L_c-x_c-14) (L_c-x_c)/4]; otherwise.

Using these equations, with L_x = 48’, the following values are obtained

| Section | x_c (‘) | V (k) | M (k’)
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>0</td>
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<td>0.00</td>
</tr>
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<td>L</td>
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<tr>
<td>M</td>
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<td>25.20</td>
<td>403.24</td>
</tr>
<tr>
<td>N</td>
<td>24</td>
<td>16.81</td>
<td>432.89</td>
</tr>
</tbody>
</table>

The ‘cantilever span’ I(R)-J-K

V_{LL+IMP}(x_c) = 23.72 [2.25 - x_c + (28-x_c)/4]/L_x]

M_{LL+IMP}(x_c) = (23.72 x_c) [2.25 - 21/L_x]

Using these equations, with L_x = 48’, the following values are obtained

| Section | x_c (‘) | V (k) | M (k’)
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>I (R)</td>
<td>16</td>
<td>51.89</td>
<td>-687.88</td>
</tr>
<tr>
<td>J</td>
<td>8</td>
<td>47.93</td>
<td>-343.94</td>
</tr>
</tbody>
</table>

The ‘end span’ A-B-C-D-E-G-H-I(L)

Here the results for x_c ≤ L_x/2 will be calculated and symmetry will be used for the other half.

For positive shear and moment

V^\star_{LL+IMP}(x_c) = (23.72/L_x) [(L_c-x_c) + (L_c-x_c-14) + (L_c-x_c-28)/4] ≥ 0

M^\star_{LL+IMP}(x_c) = x_c V^\star_{LL+IMP}(x_c); if x_c ≤ L_x/3,

= (23.72/L_x) [x_c (L_c-x_c) + x_c (L_c-x_c-14) + (L_c-x_c-14) (L_c-x_c)/4]; otherwise.

For negative shear and moment

V^-_{LL+IMP}(x_c) = - (23.72/L_x) [x_c + (x_c-14) + (x_c-28)/4] ≤ 0

or = - (23.72 L_x/L_c) [1 + (1-14/L_c) + (1-28/L_c)/4]

M^-_{LL+IMP}(x_c) = - (23.72 x_c L_x/L_c) [1 + (1-14/L_c) + (1-28/L_c)/4]

Using the derived equations, with L_c = 16’ and L_x = 64’, the following values are obtained

| Section | x_c (‘) | V^\star (k) | M^\star (k’) | V^- (k) | M^- (k’)
<table>
<thead>
<tr>
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<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
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<td>0.00</td>
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<td>311.33</td>
<td>-10.75</td>
<td>-85.99</td>
</tr>
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<td>32.24</td>
<td>515.91</td>
<td>-10.75</td>
<td>-171.97</td>
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<td>-257.96</td>
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<td>646.37</td>
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<td>-343.94</td>
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<td>*</td>
<td>624.13</td>
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<td>-429.93</td>
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<tr>
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<td>48</td>
<td>*</td>
<td>515.91</td>
<td>-32.24</td>
<td>-515.91</td>
</tr>
<tr>
<td>H</td>
<td>56</td>
<td>*</td>
<td>311.33</td>
<td>-38.92</td>
<td>-601.90</td>
</tr>
<tr>
<td>I(L)</td>
<td>64</td>
<td>*</td>
<td>0.00</td>
<td>-45.59</td>
<td>-687.88</td>
</tr>
</tbody>
</table>
4. Combination of Dead and Live Loads

The dead load and (live load + Impact) shear forces and bending moments calculated earlier at various sections of the bridge are now combined to obtain the design (maximum positive and/or negative) shear forces and bending moments.

[These calculations can be conveniently done in EXCEL, and subsequent columns should be kept for shear & flexural design]

Table 4.1 Combination of DL & LL+IMP to get $V_{Design}$ and $M_{Design}$

<table>
<thead>
<tr>
<th>Section</th>
<th>$V_{DL}$ (k)</th>
<th>$V_{LL+IMP}$ (k)</th>
<th>$V_{Design}$ (k)</th>
<th>$M_{DL}$ (k')</th>
<th>$M_{LL+IMP}$ (k')</th>
<th>$M^+_{Design}$ (k')</th>
<th>$M^-_{Design}$ (k')</th>
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</thead>
<tbody>
<tr>
<td>A</td>
<td>27.40</td>
<td>45.59 -10.75</td>
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<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
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<td>18.32</td>
<td>38.92 -10.75</td>
<td>57.24</td>
<td>182.84</td>
<td>311.33 -85.99</td>
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<td>-518.39</td>
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<td>-38.92</td>
<td>-78.39</td>
<td>-274.39</td>
<td>311.33 -601.90</td>
<td>36.94</td>
<td>-876.29</td>
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<td>-45.59</td>
<td>-97.21</td>
<td>-638.72</td>
<td>-687.88</td>
<td>0.00</td>
<td>-1326.60</td>
</tr>
<tr>
<td>I(R)</td>
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5. Design of Interior Girders

Shear Design

The shear design of interior girders is performed by using the conventional shear design equations of RCC members. The stirrup spacing is given by the equation

\[ S_{\text{(req)}} = A_s f_s d / (V - V_c) \]

where \( f_s = 20 \text{ ksi} \). If 2-legged #5 stirrups are used, \( A_s = 0.62 \text{ in}^2 \).

\[ V_c = 0.95 \sqrt{f_c b d} = 0.95 \sqrt{(0.003) 15 d} = 0.7805 d \]

\[ S_{\text{(req)}} = 12.4 d / (V - 0.7805 d) \]

\[ d_{\text{(req)}} = V / (2.95 \sqrt{f_c b}) = V / 2.4237 \]

where \( d \) and \( V \) vary from section to section.

ACI recommends that the maximum stirrup spacing \( (S) \) shouldn’t exceed \( d/2, \) or 24” or \( A_s / 0.0015b = 0.62 / 0.0225 = 27.56” \)

The calculations are carried out in tabular form and listed below.

It is convenient to perform these calculations in EXCEL.

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**Flexural Design**

The flexural design of interior girders is performed by using the conventional flexural design equations for singly/doubly reinforced RCC members (rectangular or T-beam section).

For positive moments, the girders are assumed singly reinforced T-beams with

\[ A_s = M/(f_s (d-t/2)) \]

However, the compressive stresses in slab should be checked against \( f_c (= 1.2 \text{ ksi here}) \).

For negative moments, the girders are rectangular beams. For singly reinforced beams, the depth \( d_{(req)} = \sqrt{M/R_b} \), and the required steel area \((A_s)\) at top is

\[ A_s = A_{s1} + A_{s2} = M_1/(f_s j d) + M_2/(f_s (d-d')) \]

Here \( d' \) is the depth of compression steel from the compression edge of the beam. In addition, compressive steels are necessary (in the compression zone at bottom), given by

\[ A_s' = M_2/[f_s' (d-d')] \], where \( f_s' = 2 f_s (k-d'/d)/(1-k) \leq f_s \)

Development Length of #10 bars = 0.04×1.27×40/0.03 = 51.94"

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6. Design of Articulation

The width of the girder will be doubled at the articulation; i.e., \( b_a = 30'' \), the gradual widening will start at a distance = \( 6b = 90'' \). The design parameters are the following,

- Weight of the cross-girder = \( 0.15 \times 2 \times (50.8''/12) \times 5.75'' = 7.30 \) k
- Design shear force = \( V_K = (71.66 + 7.30) \) k = 78.96 k
- Length of the articulation, \( A_L = 2' \); \(: Design moment M_{K(a)} = 78.96 \times 2'/2 = 78.96 \) k'

A bearing plate or pad will be provided to transfer the load.

- Assume bearing strength = 0.5 ksi \( \Rightarrow \) Required bearing area = 78.96/0.5 = 157.92 in²
  - The bearing area is \((12'' \times 16'')\), with thickness = 6" (assumed for pad)

The depth of girder at K is = 50.8''

- Design depth at articulation = \((50.8''-6)/2 = 22.4''\) \( \Rightarrow \) Effective depth \( d_K = 19.4'' \)

The required depth from shear \( d_{(req)} = V/(2.95\sqrt{f'_c b_u}) = 16.29'', \) which is <19.4'', OK.

- Stirrup spacing, \( S_{(req)} = A_s f_s d/(V-V_c) \)
  - where \( f_s = 20 \) ksi. If 2-legged #5 stirrups are used, \( A_s = 0.62 \) in².
  - \( V_c = 0.95\sqrt{f'_c b d_K} = 0.95\sqrt{(0.003)30 \times 19.4} = 1.561 \) d\( K \)
  - \( \Rightarrow S_{(req)} = 12.4 \times 19.4/(76.22-1.561\times19.4) = 4.94'' \)
  - \( \Rightarrow \) Provide 2-legged #5 stirrups @4.5"/c/c

  In addition, inclined bars will be provided for the diagonal cracks. These will be the same size as the main bars and their spacing will be governed by \( d/2 \) (of the main girder).

  - Here, \( d = 44.3'' \Rightarrow \) Spacing = 22.15"

  Since the length of articulation is 2' = 24'', provide 2 #10 bars @ 12"/c/c

The required depth from bending, \( d_{(req)} = \sqrt{M/R_b} = 13.04'' \), which is <19.4''

- Singly reinforced section, with required steel,

  - \( A_s = M/(f_s j d) = 78.96 \times 12/(20 \times 0.883 \times 19.4) = 2.77 \) in²

These will be adjusted with the main reinforcements in design.
Fig. 1: Design for Shear

Fig. 2: Design for Moment

Sections

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#5 Stirrup Spacing

- 8"
- 13"
- 16"

X-girders at A, D, I, K, N

Internal Hinge

Section D

- 4 #10 Bars
- #5 @ 14” c/c
- #5 Bars
- 16 #10 Bars

Section I

- 12 #10 Bars
- #5 @ 14” c/c
- 6 #10 Bars
A_s = (200/40000) \times 12 \times 44.3 \\
= 2.66 \text{ in}^2

Provide 2 #10 Bars 
(Both top & bottom)
- #10 Bars
- #5 Bars

Cross-Girder at Articulation
7. Design of Railings and Kerb

The following arrangement is chosen for the railing

Assume (for the assignment)
- Span of Railing, $S_r = L_s / 8$
- Width (b) of railing section = $(S_r + 2)$ in
- Height of Railpost = $S_r / 4 + S_r / 4 + S_r / 6$
- Width (b) of railpost section = $(S_r + 4)$ in

Railing

The assumed load on each railing = 5 k

:: Design bending moment $M_{(\pm)} = 0.8 \ (PL / 4) = 0.8 \ (5 \times 6 / 4) = 6.0 \ k'$

If the width $b = 8''$, $d_{(req)}$ from bending = $\sqrt{(M_{(\pm)} / Rb)} = \sqrt{6 \times 12 / (0.197 \times 8)} = 6.76''$

Shear force $V = 5.0 \ k$ $\Rightarrow$ $d_{(req)}$ from shear = $V / 2.95 \sqrt{f'_c} b = 5.0 / (2.95 \sqrt{0.003} \times 8) = 3.87''$

:: Assume $d = 7''$, $h = 8.5''$

:: $A_s = M_{(\pm)} (f_{jd}) = 6 \times 12 / (20 \times 0.883 \times 7) = 0.59 \ in^2$; i.e., use 2 #5 bars at top and bottom

$V_c = 0.95 \sqrt{f'_c} b d = 0.95 \sqrt{(0.003) \times 8 \times 7} = 2.91 \ k$

:: Spacing of 2-legged #3 stirrups, $S_{(req)} = A_s f_s \ d / (V - V_c) = 0.22 \times 20 \times 7 / (5.0 - 2.91) = 14.76''$

:: Provide 2-legged #3 stirrups @3.5"c/c (i.e., d/2)
**Rail Post**

Design bending moment $M_{(-)} = 5 \times 1.5 + 5 \times 3.0 = 22.5 \text{ k}'$

If the width $b = 10''$, the $d_{(req)}$ from bending $= \sqrt{(M_{(-)}/Rb)} = \sqrt{\{22.5 \times 12/(0.197 \times 10)\}} = 11.71''$

Shear force $V = 10.0 \text{ k}$

$\Rightarrow d_{(req)}$ from shear $= V/2.95\sqrt{f_c} b = 10.0/(2.95\sqrt{(0.003) \times 10}) = 6.19''$

$\therefore$ Assume $d = 12''$, $h = 13.5''$

$\therefore A_s = M_{(-)}/(f_{jd}) = 22.5 \times 12/(20 \times 0.883 \times 12) = 1.27 \text{ in}^2$; i.e., use 3 #6 bars inside

$V_c = 0.95\sqrt{f_c} bd = 0.95\sqrt{(0.003) \times 10 \times 12} = 6.24 \text{ k}$

If 2-legged #3 stirrups are used, $A_s = 0.22 \text{ in}^2$

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_p d/(V - V_c) = 0.22 \times 20 \times 12/(10.0 - 6.24) = 14.06''$

$\therefore$ Provide 2-legged #3 stirrups @6" c/c (i.e., d/2)
**Edge Slab and Kerb**

**Edge Slab**

Design load = 16 k, assumed width = 4 ft

Design bending moment for edge slab \( M_{(-)} = 16/4 \times 18/12 = 6.0 \text{ k}'/\text{ft} \)

\[
d_{(req)} = \sqrt{(M_{(-)}/Rb)} = \sqrt{(6.0/0.197)} = 5.52''
\]

Assuming \( d = 5.5'' \), \( t = 7.5'' \)

\[\therefore A_s = M_{(-)}/(f_j d) = 6.0 \times 12/(20 \times 0.883 \times 5.5) = 0.74 \text{ in}^2/\text{ft},\]

which is greater than the reinforcement (\( = 0.54 \text{ in}^2/\text{ft} \)) for the main slab.

There are two alternatives, using

(a) \( d = 5.5'' \), \( t = 7'' \), with \( #6 @10'' \text{ c/c} \) (like main slab) + one extra \( #6 \) after 2 main bars

(b) \( d = 7.5'' \), \( t = 9'' \), with \( #6 @10'' \text{ c/c} \) (like main slab)

Use \( A_{s(temp)} = 0.03t = 0.21 \text{ in}^2/\text{ft}, \text{ or } 0.27 \text{ in}^2/\text{ft}, \text{ i.e., } #5 @14'' \text{ c/c or } 12'' \text{ c/c transversely} \)

**Kerb**

Design load = 10 k/4' ⇒ Design bending moment for kerb \( M_{(-)} = 10/4 \times 10/12 = 2.08 \text{ k}'/\text{ft} \)

\[\therefore \text{The required depth, } d_{(req)} = \sqrt{(M_{(-)}/Rb)} = \sqrt{(2.08/0.197)} = 3.25'' << \text{ assumed } d = 20'', \text{ t = 24''} \]

\[\therefore A_s = M_{(-)}/(f_j d) = 2.08 \times 12/(20 \times 0.883 \times 20) = 0.071 \text{ in}^2/\text{ft}, \text{ which is not significant} \]

\[\therefore A_{s(temp)} = 0.03 h = 0.03 \times 17.5 = 0.525 \text{ in}^2/\text{ft} \]

\[\therefore \text{Provide } #6 \text{ bars } @10'' \text{ c/c over the span (i.e., consistent with } #6 \text{ bars } @10'' \text{ c/c for the slab)} \]
and \( = 0.525 \times 24/12 = 1.05 \text{ in}^2; \text{ i.e., } 4 \#5 \text{ bars within the width of kerb.} \)
8. Design of Substructure

Design of Abutment and Wing Walls

Stability Analysis

The effect of front soil is ignored and wall geometry is simplified (denoted by the dotted lines).

**Sliding**

Approximate Vertical load = \(12.69 + 0.12\times\{(3+14)\times8 + 4\times8.75\} + 0.15\times\{14.5\times2 + 2\times14\}\)

= \(12.69 + 16.32 + 4.2 + 4.35 + 4.2 = 41.76\) k/′

∴ Horizontal resistance \(H_R = 0.45\times41.76 = 18.79\) k/′

Horizontal load \(H = 1.90 + 0.12\times3\times20 + 0.12\times20/3\times20/2 = 1.90 + 2.4 + 8.0 = 12.30\) k/′

∴ Factor of Safety against sliding = \(H_R/H = 18.79/12.30 = 1.53 > 1.5,\ OK\)

**Overturning**

Approximate resisting moment

\(M_R = 12.69\times(4.5+0.625) + 16.32\times (6.5+4) + 4.2\times(5.75+4.375)+ 4.35\times7.25 + 4.2\times5.5\)

= \(336.18\) k′/′

Overturning moment \(M = 1.90\times16 + 2.4\times20/2 + 8.0\times20/3 = 107.78\) k′/′

∴ Factor of Safety against overturning = \(M_R/M = 336.18/107.78 = 3.12 > 1.5,\ OK\)

For the assignment Multiply all dimensions by Factor \(F = L/80\)
Design of Back-wall

Design wheel load = 16 k and assumed loaded length ≈ 4'

:. Load per unit width \( V = 16/4 = 4 \text{ k}'' \)

Moment per unit width \( M = 4\times 2/2 = 4 \text{ k}''/\text{'} \)

:. \( d_{(\text{req})} \) for moment = \( \sqrt{(4/0.186)} = 4.64'' \)

:. \( d = 18 - 3 = 15'' > d_{(\text{req})} \)

\( A_s = 4\times 12/(20\times 0.883\times 15) = 0.18 \text{ in}^2/\text{'} \)

\( A_{s_{(\text{temp})}} = 0.03\times 18 = 0.54 \text{ in}^2/\text{'} \)

This should be adjusted with stem reinforcement.

Design of Stem

Design \( V/\text{length} = 1.9 + 0.12\times 18 + 0.72\times 18/2 = 1.9 + 2.16 + 6.48 = 10.54 \text{ k}''/\text{'} \)

Design \( M/\text{length} = 1.9\times 14 + 2.16\times 18/2 + 6.48\times 18/3 = 84.92 \text{ k}''/\text{'} \)

:. \( d_{(\text{req})} \) for shear = \( 10.54/(0.95\times \sqrt{(0.003\times 12)}) = 16.88'' \)

and \( d_{(\text{req})} \) for moment = \( \sqrt{(84.92/0.186)} = 21.38'' \)

\( d = 24 - 3.5 = 20.5'' < d_{(\text{req})} \Rightarrow t = 25'', d = 21.5'' \)

\( A_s = 84.92\times 12/(20\times 0.883\times 21.5) = 2.68 \text{ in}^2/\text{'} \)

:. Use #10 @ 5.5'' \text{ c/c} \ (\text{along the length near soil})

\( A_{s_{(\text{temp})}} = 0.03\times 25 = 0.75 \text{ in}^2/\text{'} \)

:. Use #5 @ 5'' \text{ c/c} \ (\text{along the width and length farther from soil})

This should also be adequate for the back-wall

(both along length and width)

Design of Toe and Heel

Total vertical force on the soil below the wall = 41.76 \text{ k}''/\text{'}

The resultant moment about the far end to the toe = \( M_{R-M} = 336.18 - 107.78 = 228.40 \text{ k}''/\text{'} \)

:. \( e_0 = 228.40/41.76 = 5.47'' \), which is >14.5/3 and <14.5\times 2/3 \Rightarrow \text{uplift avoided.} \)

The maximum soil pressure = \( (41.76/14.5)\times [1+\text{6}(14.5/2-5.47)/14.5] = 5.00 \text{ ksf} \), which is > 2 \text{ ksf.} \)

:. Pile foundation is suggested, and the toe and heel should be designed as pile cap.
Design of Piles and Pile-cap
The following arrangement of piles is assumed for the width of the toe and heel (i.e., 14.5') and within the c/c distance of girders (i.e., width = 5.75').

Pile Forces
For the total width of one row of piles, \( V = 41.76 \text{ k}/5.75' = 240.10 \text{ k} \)
and \( M = (41.76 \times 7.25 - 228.40) \text{ k}'/5.75' = 483.19 \text{ k}' \)
Pile reactions are given by \( F_i = \frac{V}{n} + \frac{M y_i}{\sum y_i^2} \)
where \( n = \) Number of piles = 4, \( \sum y_i^2 = 2 \times 1.75^2 + 2 \times 5.25^2 = 61.25 \text{ ft}^2 \)
∴ Here, \( F_1 = 240.10/4 + 483.19 y_1/61.25 = 60.03 + 7.89 y_1 \)
∴ \( F_1 = 60.03 + 7.89 \times 5.25 = 101.44 \text{ k}, F_2 = 60.03 + 7.89 \times 1.75 = 73.83 \text{ k} \)
\( F_3 = 60.03 + 7.89 \times (-1.75) = 46.22 \text{ k}, F_4 = 60.03 + 7.89 \times (-5.25) = 18.61 \text{ k} \)

Design of Piles
Using 3% steel, \( P = 0.85 (0.25 f'_c + 0.03 f_s) \pi/4 D^2 \)
\( \Rightarrow 101.44 = 0.85 (0.25 \times 3 + 0.03 \times 20) \pi/4 D^2 \Rightarrow D = 10.61'' \)
∴ Use 11” diameter piles with 6 #5 bars and #5 ties @11” c/c
Smaller sections can be used for the piles other than Pile 1
The piles should be long enough to transfer the axial loads safely to the surrounding soil.
Assuming the entire pile load for pile 1 to be resisted by skin friction, \( F_1 = \alpha_2 \pi D L q_a/2 \)
where \( \alpha_2 = \) Reduction factor for soil disturbance \( \approx 0.8, D = \) Pile diameter = 11”
\( L = \) Pile length, \( q_a = \) Allowable compressive stress on soil = 2 ksf
∴ \( 101.44 = 0.8 \times (\pi \times 11/12 \times L) \times 2/2 \Rightarrow L = 44.03', \) \( \therefore \) Provide 45’ long piles.
Design of Pile-cap

For the design of pile-cap, the pile loads per girder are considered along with the soil pressure converted to soil load per girder width. The pile loads on the pile-cap for c/c girder width (5.75’) are shown below.

1. Design of Toe

The assumed thickness = 2’ = 24”.

The pile-cap is assumed to have 3” effective cover and 6” embedment for piles.

Maximum Punching shear = 101.44 k, allowable punching stress = 1.9×√(0.003) = 0.104 ksi

Punching area around a 12” pile = π (11+d) d = 101.44/0.104 ⇒ d_(req) = 13.3”; i.e., t_(req) = 22.5”

Maximum flexural shear = 101.44 k, allowable shear stress = 0.95×√(0.003) = 0.052 ksi

Shearing area = 5.75×12 d = 101.44/0.052 ⇒ d_(req) = 28.25”; i.e., t_(req) = 37.25”

Maximum bending moment = 101.44×2.5 = 253.60 k’

⇒ d_(req) = √{253.60/(0.186×5.75)} = 15.41”; i.e., t_(req) = 24.41”

∴ t = 38”, d = 29” ⇒ A_s = 253.60×12/(20×0.883×29) = 5.94 in^2; i.e., 5.94/5.75 = 1.03 in^2/ft

A_{s(temp)} = 0.03×38 = 1.14 in^2/ft

2. Design of Heel

Maximum flexural shear = 14.49×7– 46.22 –18.61 = 36.60 k, allowable shear stress = 0.052 ksi

Shearing area = 5.75×12 d = 36.60/0.052 ⇒ d_(req) = 10.19”; i.e., t_(req) = 19.19”

Maximum bending moment = -14.49×8×8/2 + 46.22×2.5 + 18.61×6 = -236.47 k’

⇒ d_(req) = √{236.47/(0.186×5.75)} = 14.88”; i.e., t_(req) = 23.88”

∴ t = 38”, d = 29” ⇒ It will be OK for punching shear also

∴ A_s = 236.47×12/(20×0.883×29) = 5.54 in^2; i.e., 0.96 in^2/ft, A_{s(temp)} = 0.03×38 = 1.14 in^2/ft
Abutment Reinforcements

- #5 @ 5" c/c
- #10 @ 5.5" c/c, alt. stopped at 9' from bottom
- #5 @ 5" c/c
- #10 @ 12" c/c, alt. stopped

Pile Length = 45'

Pile Reinforcements

- 6 #6 bars, with #5 ties @ 11" c/c
5. Design of Interior Girders (USD)

Shear Design

The shear design of interior girders is performed by using the conventional shear design equations of RCC members. The stirrup spacing is given by the equation

\[ S_{\text{req}} = \frac{A_s f_y d}{V_u/\phi - V_c} \]

where \( f_y = 40 \text{ ksi} \). If 2-legged #5 stirrups are used, \( A_s = 0.62 \text{ in}^2 \).

\[ V_c = 1.9\sqrt{f_c' b_d} = 1.9\sqrt{(0.003) 15 d} = 1.561 d \]

\[ S_{\text{req}} = 24.8 \frac{d}{(V_u/0.85 - 1.561 d)} \]

\[ d_{\text{req}} = \frac{V}{(5.9\phi\sqrt{f_c' b_w})} = V/4.120 \]

where \( d \) and \( V \) vary from section to section.

ACI recommends that the maximum stirrup spacing \( (S) \) shouldn’t exceed \( d/2 \), or 24” or \( A_s f_y/0.06b_w = 24.8/0.90 = 27.56” \)

The calculations are carried out in tabular form and listed below.

It is convenient to perform these calculations in EXCEL.

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<th>Section</th>
<th>x from left (&quot;)</th>
<th>h (&quot;)</th>
<th>d (&quot;)</th>
<th>( V_u ) (kips)</th>
<th>( d_{\text{req}} ) (&quot;)</th>
<th>( S_{\text{req}} ) from formula (&quot;)</th>
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Flexural Design

\[ p_{\text{max}} = (0.75 \alpha f_y / f_y) [87/(87+f_y)] = 0.0277, \quad R_u = \phi p_{\text{max}} f_y [1-0.59p_{\text{max}} f_y/f_y] = 0.781 \text{ ksi} \]

The section shown is chosen for all the beams.

\[ d = 2.5'' \]

(iii) \( c/c = 69'' \)

\[ d_{15} = 4.40 \]

\[ d = 4.40 \]

\[ d = 4.40 \]

\[ b_s = 15'' \]

For all the beams.

\[ A_{s(+)} = (f_c/f_y)[(1-1/2M_c/(\phi_c b_y d^2))]b_y d \]

\[ (2.55/40) \times [1-1/2M_c/(0.9\times 2.55 \times 15 \times d^2)] 	imes 15 \times d \]

\[ 0.956d [1-1/0.697M_c/d^2] \]

These calculations, performed in EXCEL, are listed below.

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Fig. 1: Design for Shear (USD)

Fig. 2: Design for Moment (USD)
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- #5 Stirrup Spacing: 8" 13" 16" 20" 16"
- X-girders at A, D, I, K, N
- Internal Hinge

- #5 @ 16" c/c
- #5 Bars
- 14 #10 Bars

Section D

- 4 #10 Bars

Section I

- 10 #10 Bars
- #5 @ 16" c/c
- 4 #10 Bars
6. Design of Articulation (USD)

The width of the girder will be doubled at the articulation; i.e., \( b_a = 30" \), the gradual widening will start at a distance = \( 6b = 90" \). Length of the articulation, \( A_L = 2' \).

Reaction from interior girder = 113.22 k

Weight of the cross-girder = \( 0.15 \times 2 \times (50.8"/12) \times 5.75' \times 1.4 = 10.23 \) k

Design shear force = \( V_K = (113.22 + 10.23) \) k = 123.45 k

\[ \therefore A_L = 2' \Rightarrow \text{Design moment } M_{K(a)} = 123.45 \times 2'/2 = 123.45 \text{ k}' \]

A bearing plate or pad will be provided to transfer the load.

Assume bearing strength = 1.0 ksi \( \Rightarrow \) Required bearing area = 123.45/1.0 = 123.45 in\(^2\)

\[ \therefore \text{The bearing area is (12"\times12"), with thickness = 6" (assumed for pad)} \]

The depth of girder at K is = 50.8"

\[ \therefore \text{Design depth at articulation} = (50.8 - 6)/2 = 22.4" \Rightarrow \text{Effective depth } d_K = 19.4" \]

The required depth from shear \( d_{(req)} = V_K/(5.9\sqrt{f'_c b_a}) = 12.73" \), which is <19.4", OK.

\[ \therefore \text{Stirrup spacing, } S_{(req)} = A_s f_y d/(V_u/\phi - V_c) \]

If 2-legged #5 stirrups are used, \( A_s = 0.62 \text{ in}^2 \).

\[ V_c = 1.9\sqrt{f'_c b d_K} = 1.9\sqrt{(0.003) 30 d_K} = 3.122 d_K \]

\[ \Rightarrow S_{(req)} = 24.8 d_K/(V_K/0.85 - 3.122 d_K) = 24.8 \times 19.4/(145.23 - 3.122 \times 19.4) = 5.68" \]

\[ \therefore \text{Provide 2-legged #5 stirrups @5.5"c/c} \]

In addition, inclined bars will be provided for the diagonal cracks. These will be the same size as the main bars and their spacing will be governed by \( d/2 \) (of the main girder).

Here, \( d = 44.3" \Rightarrow \text{Spacing} = 22.15" \)

Since the length of articulation is \( 2' = 24" \), provide 2 #10 bars @ 12"c/c

The required depth from bending, \( d_{(req)} = \sqrt{(M_{K(a)}/R_u b_a)} = 7.95" \), which is <19.4"

\[ \Rightarrow \text{Singly reinforced section, with required steel,} \]

\[ A_s = (f_c/f_y)[1-\sqrt{1-2M_{K(a)}/(\phi f_c b_a d^2)}] \text{ } b_a d = 2.19 \text{ in}^2 \]

These will be adjusted with the main reinforcements in design.
#5 @ 5.5" c/c
#5 @ 16" c/c

2 #10 Diagonal Bars

#5 @ 16" c/c
#5 @ 5.5" c/c

Longitudinal Section of Articulation

\[
A_s = \left(\frac{200}{40000}\right) \times 12 \times 44.3 = 2.66 \text{ in}^2
\]

Provide 2 #10 Bars
(Both top & bottom)
- #10 Bars
- #5 Bars

Cross-Girder at Articulation
7. Design of Railing and Rail Post (USD)

The following arrangement is chosen for the railing

![Diagram of railing and rail post]

**Railing**

The assumed load on each railing = 5 k

\[
\therefore \text{Design bending moment } M_{(\pm)} = 0.8 \times (PL/4) = 0.8 \times (1.7\times5\times6/4) = 10.2 \text{ k'}
\]

If the width \( b = 6'' \), the required depth from bending is

\[
d_{(\text{req})} = \sqrt{M_{(\pm)}/R_u b} = \sqrt{10.2\times12/(0.781\times6)} = 5.11''; \text{ i.e., assume } d = 6'', \ t = 7.5''
\]

\[
\therefore A_s = (f_c/f_y)[1-\sqrt{1-2M_{(\pm)}/(\phi f_c bd^2)}] \times bd = 0.662 \text{ in}^2; \text{ i.e., use 2 #6 bars at top and bottom}
\]

Shear force \( V_u = 1.7 \times 5 = 8.5 \text{ k}, \ V_c = 1.9\sqrt{f'_c bd} = 1.9\sqrt{(0.003) \times 6 \times 6} = 3.75 \text{ k}
\]

If 2-legged #3 stirrups are used, \( A_s = 0.22 \text{ in}^2 \)

\[
\therefore \text{Stirrup spacing, } S_{(\text{req})} = A_s f_y d/(V_u/\phi - V_c) = 0.22\times40\times6/(8.5/0.85-3.75) = 8.44''
\]

\[
\therefore \text{Provide 2-legged #3 stirrups @3'' c/c (i.e., d/2)}
\]
**Rail Post**

Design bending moment $M_{(-)} = 1.7 \times 5 \times 1.5 + 1.7 \times 5 \times 3.0 = 38.25$ k

If the width $b = 6''$, the required depth from bending is

$$d_{(req)} = \sqrt{(M_{(-)}/R_u b)} = \sqrt{38.25 \times 12/(0.781 \times 8)} = 8.57''$$

i.e., assume $d = 10''$, $t = 11.5''$

$\therefore A_s = (f_c/f_y)[1-\sqrt{1-2M_{(-)}/(\phi f_c b d^2)}]$ bd = 1.494 in$^2$; i.e., use 3 #6 bars inside

Shear force $V_u = 1.7 \times 10 = 17$ k, $V_c = 1.9\sqrt{f_c} b d = 1.9\sqrt{(0.003)} \times 8 \times 10 = 8.33$ k

If 2-legged #3 stirrups are used, $A_s = 0.22$ in$^2$

$\therefore$ Stirrup spacing, $S_{(req)} = A_s f_y d/(V_u/\phi - V_c) = 0.22 \times 40 \times 10/(17/0.85 - 8.33) = 7.54''$

$\therefore$ Provide 2-legged #3 stirrups @5" c/c (i.e., d/2)
Kerb

Design bending moment \( M_{(-)} = 1.7 \times 5/4 \times (12/12) = 2.13 \text{k'ft} \)

If the width \( b = 12'' \), the required depth from bending is
\[
d_{(req)} = \sqrt{(M_{(-)}/Reb)} = \sqrt{[2.13 \times 12/(0.781 \times 12)]} \approx \text{assumed } d = 7.5'', \ t = 9''\]
\[
\therefore A_s = (f_c/f_y) [1 - \sqrt{1 - 2M_{(-)}/(\phi f_c bd^2)}] \ bd = \text{in}^2; \ i.e., \ use \ 3 \ #6 \ bars \ inside
\]
Shear force \( V_u = 1.7 \times 10 = 17 \text{k}, \ V_c = 1.9 \phi f_c bd = 1.9 \phi (0.003) \times 8 \times 10 = 8.33 \text{k} \)
If 2-legged #3 stirrups are used, \( A_s = 0.22 \text{ in}^2 \)
\[
\therefore \text{Stirrup spacing, } S_{(req)} = A_s f_y d/(V_u \phi - V_c) = 0.22 \times 40 \times 10/(17/0.85 - 8.33) = 7.54''
\]
\[
\therefore \text{Provide 2-legged #3 stirrups @5''c/c (i.e., } d/2)\]
8. Design of Substructure

Design of Abutment and Wing Walls

Stability Analysis (Using Working Load and Allowable Pressure)

The effect of front soil is ignored and wall geometry is simplified (denoted by the dotted lines).

**Sliding**

Approximate Vertical load = 12.69 + 0.12×{(3+14)×8 + 4×8.75} + 0.15×{14.5×2 + 2×14}

= 12.69 + 16.32 + 4.2 + 4.35 + 4.2 = 41.76 k/′

\[ \therefore \text{Horizontal resistance } H_R = 0.45 \times 41.76 = 18.79 \text{ k/′} \]

Horizontal load \( H = 1.90 + 0.12 \times (3 \times 20 + 20 \times 20/2)/3 = 1.90 + 2.4 + 8.0 = 12.30 \text{ k/′} \)

\[ \therefore \text{Factor of Safety against sliding } = H_R/H = 18.79/12.30 = 1.53 > 1.5, \text{ OK} \]

**Overturning**

Approximate resisting moment

\[ M_R = 12.69 \times (4.5+0.625) + 16.32 \times (6.5+4) + 4.2 \times (5.75+4.375) + 4.35 \times 7.25 + 4.2 \times 5.5 \]

\[ = 329.36 \text{ k/′/′} \]

Overturning moment \( M = 1.90 \times 16 + 2.4 \times 20/2 + 8.0 \times 20/3 = 107.73 \text{ k/′/′} \)

\[ \therefore \text{Factor of Safety against overturning } = M_R/M = 329.36/107.73 = 3.06 > 2.5, \text{ OK} \]
Design of Back-wall

Design wheel load = 16×1.7 = 27.2 k, assumed loaded length = 4’

\[ V = \frac{27.2}{4} = 6.8 \text{k}’’ \]

\[ M = \frac{6.8\times15}{2} + \frac{6.8\times9}{2} = 6.8 \text{k}’’ \]

\[ d_{\text{req}} \text{ for shear} = \frac{6.8}{1.9\times0.85\times\sqrt{0.003\times12}} = 6.8/1.062 = 6.41’’ \]

\[ d_{\text{req}} \text{ for moment} = \sqrt{\frac{6.8}{0.781}} = 2.95’’ \]

\[ d = 18–3 = 15’’ \gg d_{\text{req}} \]

\[ A_s = \left(\frac{f_c}{f_y}\right)\left[1-\sqrt{1-2M/(\phi f_c bd^2)}\right] \text{bd} = 0.15 \text{ in}^2’’ \]

\[ A_{s\text{(temp)}} = 0.03\times18 = 0.54 \text{ in}^2’’ \]

This should be adjusted with stem reinforcement

Design of Stem

Design shear/length = \((0.12\times18 + 0.72\times18/2)\times1.4 = 3.02 + 9.07 = 12.09 \text{k}’’ \]

Design moment/length = \(3.02\times18/2 + 9.07\times18/3 = 81.60 \text{k}’’ \]

\[ d_{\text{req}} \text{ for shear} = \frac{12.09}{1.062} = 11.39’’ \]

\[ d_{\text{req}} \text{ for moment} = \sqrt{\frac{81.60}{0.781}} = 10.22’’ \]

\[ d = 24–3.5 = 20.5’’ \gg d_{\text{req}} \]

\[ A_s = \left(\frac{f_c}{f_y}\right)\left[1-\sqrt{1-2M/(\phi f_c bd^2)}\right] \text{bd} = 1.39 \text{ in}^2’’ \]

\[ A_{s\text{(temp)}} = 0.03\times24 = 0.72 \text{ in}^2’’ \]

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\[ A_{s\text{(temp)}} = 0.03\times24 = 0.72 \text{ in}^2’’ \]

Use #10 @ 10.5’’ c/c (along the length near soil)

\[ A_{s\text{(temp)}} = 0.03\times24 = 0.72 \text{ in}^2’’ \]

Use #5 @ 5’’ c/c (along the width and length farther from soil)

This should also be adequate for the back-wall

(both along length and width)

Design of Toe and Heel

Total vertical force on the soil below the wall = 41.76 k’’

The resultant moment about the far end to the toe = \(M_R - M = 329.36 - 107.73 = 221.63 k’’ \]

\[ e_0 = 221.63/41.76 = 5.31’’, \text{ which is } > 14.5/3 \text{ and } < 14.5\times2/3 \Rightarrow \text{uplift avoided.} \]

The maximum soil pressure = \((41.76/14.5)\times[1+6(14.5/2–5.31)/14.5] = 5.20 \text{ ksf}, \text{ which is } > 2 \text{ ksf.} \]

\[ p_h (\text{ksf}) \]

\[ A_{s\text{(temp)}} = 0.03\times24 = 0.72 \text{ in}^2’’ \]

\[ A_{s\text{(temp)}} = 0.03\times24 = 0.72 \text{ in}^2’’ \]

Pile foundation is suggested, and the toe and heel should be designed as pile cap.
Design of Piles and Pile-cap

The following arrangement of piles is assumed for the width of the toe and heel (i.e., 14.5’) and within the c/c distance of girders (i.e., width = 5.75’).

Pile Forces

Vertical force and resultant moment in USD are = 60.84 k’ and = 473.27 – 156.61 = 316.67 k’

For the total width of one row of piles, V = 60.84 k’ × 5.75’ = 349.82 k

and M = (60.84 × 7.25 – 316.67) k’ × 5.75’ = 715.38 k

Pile reactions are given by \( F_i = V/n + M y_i/\sum y_i^2 \)

where \( n = \) Number of piles = 4, \( \sum y_i^2 = 2 \times 1.75^2 + 2 \times 5.25^2 = 61.25 \text{ ft}^2 \)

\[ . \] Here, \( F_i = 349.82/4 + 715.38 \frac{y_i}{61.25} = 87.46 + 11.68 \frac{y_i}{y_i} \)

\[ . \] \( F_1 = 87.46 + 11.68 \times 5.25 = 148.77 \text{ k}, F_2 = 87.46 + 11.68 \times 1.75 = 107.90 \text{ k} \)

\( F_3 = 87.46 + 11.68 \times (-1.75) = 67.02 \text{ k}, F_4 = 87.46 + 11.68 \times (-5.25) = 26.14 \text{ k} \)

Design of Piles

For the load \( F_1, 148.77 = 0.80\phi A_g\{f_c + p (f_y - f_c)\} = 0.80 \times 0.70 A_g \{2.55 + 0.03 \times (40 - 2.55)\} \)

\[ . \] \( A_g = 72.32 \text{ in}^2 \Rightarrow D = 9.60”\); i.e., Provide 10”-dia piles with 4 #6 bars and #3 ties @10” c/c

The same pile will be used for all piles though smaller sections can be used for the others

The piles should be long enough to transfer the axial loads safely to the surrounding soil by skin friction and end bearing.

Assuming the entire pile load for Pile1 to be resisted by skin friction, \( F_{1(WSD)} = \alpha_2 (\pi DL) q_a/2 \)

where \( \alpha_2 = \) Reduction factor for soil disturbance \( \cong 0.8\), \( D = \) Pile diameter = 10” = 0.833’, \( L = \) Pile length, \( q_a = \) Allowable compressive stress on soil = 2 ksf

\[ . \] \( 100.02 = 0.8 \times (\pi \times 0.83 \times L) \times 2/2 \Rightarrow L = 47.76’\); ; Provide 48’ long piles
Design of Pile-cap

For the design of pile-cap, the pile loads per girder are considered along with the soil pressure converted to soil load per girder width. The pile loads on the pile-cap for c/c girder width (5.75’) are shown below.

1. Design of Toe

The pile-cap is assumed to have 3” effective cover and 6” embedment for piles.

Maximum punching shear = 148.77 k, shear strength = 3.8×0.85×√(0.003) = 0.177 ksi

Punching area around a 10” pile = π (10+d) d = 148.77/0.177 ⇒ d_{(req)} = 12.1”; i.e., t_{(req)} = 21.1”

Maximum flexural shear = 148.77 k, shear strength = 1.9×0.85×√(0.003) = 0.088 ksi

Shearing area = 5.75×12 d = 148.77/0.088 ⇒ d_{(req)} = 24.4”; i.e., t_{(req)} = 33.4”

Maximum bending moment = 148.77×2.5 = 371.93 k’

⇒ d_{(req)} = √(371.93/(0.781×5.75)) = 9.1”; i.e., t_{(req)} = 18.1”

.: t = 34”, d = 25” ⇒ A_s = (f_c/f_y)[1−√(1−2M/(f_cbd^2))] bd = 7.69 in^2; i.e., 7.69/5.75 = 1.34 in^2/ft

A_{s(temp)} = 0.03×34 = 1.02 in^2/ft

2. Design of Heel

Maximum flexural shear =20.29×7−67.02−26.14 = 48.85 k, allowable shear stress = 0.088 ksi

Shearing area = 5.75×12 d = 48.85/0.088 ⇒ d_{(req)} = 8”; i.e., t_{(req)} = 17”

Maximum bending moment = −20.29×8×8/2 + 67.02×2.5 + 26.14×6 = −324.78 k’

⇒ d_{(req)} = √(324.78/(0.781×5.75)) = 8.5”; i.e., t_{(req)} = 17.5”

.: t = 34”, d = 25” ⇒ It will be OK for punching shear also

.: A_s = (f_c/f_y)[1−√(1−2M/(f_cbd^2))] bd = 5.62 in^2; i.e., 0.98 in^2/ft, A_{s(temp)} = 0.03×34 = 1.02 in^2/ft
Abutment Reinforcements

Pile Length = 48'

Pile Reinforcements

#5 @ 5” c/c

#10 @ 10.5” c/c, alt. stopped at 9’ from bottom

#5 @ 5” c/c

#10 @ 12” c/c, alt. stopped

#10 @ 14” c/c

4 #6 bars, with #3 ties @ 10” c/c